CONCRETE PILES

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Concrete for Permanence

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FOREWORD

Piles have been used for hundreds of years. Much practical information has been accumulated by those experienced in their use, but comparatively little has been written on the subject. This has been true of concrete piles as it has of other types. The rapidly increasing use of concrete piles, however, has created an urgent demand for information pertaining to their design, manufacture and driving. To meet this demand, data from many sources have been assembled and are presented in this booklet. No effort has been made to prepare a complete treatise on the subject which has many phases. Only the fundamentals have been discussed but sufficient information is given to enable a designer to analyze pile foundations and select the most economical pile for the conditions under which it is to be used. Data are included for the design of bearing and sheet piles for both driving and service loads. The best practice in manufacturing, driving and testing piles is also presented.

CONTENTS

Large precast concrete piles are being extensively used for railroad trestles because they give the lowest cost per ton of load capacity. The high load capacity permits three piles in each bent, allowing construction with least interference to traffic. A typical installation with precast deck slabs is this one near Corning, Ark., on the Missouri Pacific RR.

Sloop Channel Bridge of the Meadowbrook Causeway. This is one of several structures for which concrete piles were selected by the Long Island State Park Commission, N. Y.

Concrete Piles

SECTION I — INTRODUCTION

Concrete piles as precast units are used to support essentially vertical loads in foundations; to carry combined vertical and lateral loads in trestles, docks and similar structures; to resist lateral loads and prevent the passage of water in bulkheads and cut-off walls.

Cast-in-place piles are used extensively in foundations to carry vertical loads.

Experience with concrete piles has demonstrated their numerous advantages for a wide variety of conditions of service.

Adaptability

Concrete piles should be considered for any location where piles are required. The choice of either precast or cast-in-place types enables the designer to use concrete piles economically to solve foundation problems where other kinds of piles could not be used.

Piles may be built in sections and assembled during driving where there is restricted head room. They may be made hollow to reduce their weight or to permit larger diameters, thus providing added skin friction area to accommodate especially unfavorable foundation conditions; or cast with connections for braces or structural members as desired. They may be made in any length required and are readily lengthened or shortened after driving.

Design

Concrete piles are "made to order." Size, length, shape, strength—almost every physical dimension and characteristic may be designed to meet manufacturing, shipping, driving, service and exposure conditions, as well as economic requirements.

Manufacture

Precast concrete piles may be cast either on the job or at a remote casting yard. The manufacturing facilities provided may be large or small, and include "homemade" equipment or the most elaborate modern machinery, depending upon the size of the project.

The finished product may be left rough, or smooth with true sharp corners, depending upon the nature of the job and whether portions of the piles will be left exposed.

For cast-in-place piles, a similarly wide range of "manufacturing" facilities, depending on the requirements of the project, is afforded.

Durability

The location of the ground water table need not be considered where concrete piles are used, because there is no deterioration at the water line if concrete is properly made.

The quality of the concrete may be chosen to meet unusual exposure conditions such as erosion from moving water, abrasion, contact with alkali soils and freezing and thawing. For special conditions a protective cover can be provided for the critical area.

Concrete piles are not damaged by termites, nor by marine borers such as the limnoria and teredo.

They are fireproof. Fires on land from grass and forests and under decks over oil-covered water are successfully resisted. Even under prolonged exposure to intense fires they are usually free from structural damage.

Field Conditions

Concrete piles may be driven in any soil to penetrations comparable to other types of piles. They are handled, unloaded and driven in essentially the same manner as are other piles. They may be readily cut off or extended where driving conditions make it necessary.

Economy

Concrete piles show marked economies when compared with other types on the logical basis of COST PER TON OF LOAD-CARRYING CAPACITY, and when other savings resulting from their use are considered. The load capacity of concrete piles can be varied at will by changing their diameter and length; hence fewer piles usually are required to carry a given load. Foundations are smaller. Cut-offs are independent of ground water level, so frequently less excavation is required and foundation walls need not be as deep. Durability under severe exposure, resistance to fire and attacks of borers result in fewer replacements and longer life. In addition, when concrete materials and equipment are on the job, the manufacture of concrete piles at the site increases the use factor for the equipment and reduces overhead cost.

SECTION II — DETERMINATION OF FOUNDATION CHARACTERISTICS

During recent years there has been an increasing amount of scientific attention devoted to the load-carrying capacity of soils based, in large measure, upon the work of Dr. Charles Terzaghi.* The structure and plastic properties of soil are now recognized as of utmost importance in designing foundations.

The load capacity of piles is affected by:

Structure, water content, frictional and cohesive properties of the soil;

Disturbance caused during construction by such operations as pile driving, drainage and removal of lateral restraint;

Later plastic yielding and water movement in supporting soil.

There are numerous and complex combinations of soil materials and arrangement of strata existing in nature. The foundation engineer, therefore, can not always have at hand definite design data to meet all the situations he encounters, but must depend to an unusual degree upon his experience and good judgment. However, certain fundamental concepts of soil types and typical arrangements of strata such as those given below are of considerable assistance.

Classifications of Soil Structure

Three general classifications of soil structure are recognized and each has certain ability to support vertical and lateral loads.

Class 1. *Bed rock or a very hard stratum within easy reach.* For this condition piles transfer vertical loads as columns directly into the hard substratum.

*"The Science of Foundations — Its Present and Future" by Charles Terzaghi; *Transactions* A.S.C.E. Vol. 93 (1929) pp. 270-301.

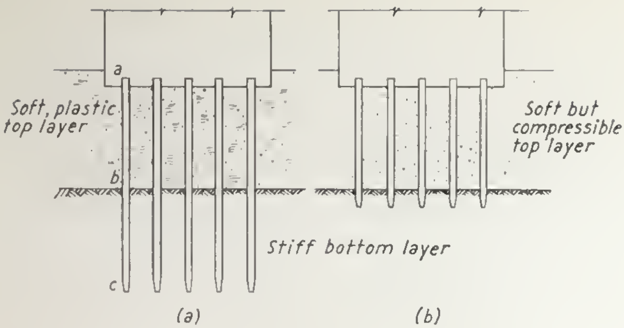


Fig. 1

Class 2. *Compressible upper soil layers with stiff layers below.* If these soft and plastic upper layers cannot be consolidated by driving piles into them, the piles must be driven far enough into the lower layers to support the entire load, as shown in Fig. 1(a). In this type soil, load tests of short duration may show that the upper layers (*a-b*) have some load-carrying value; however, plastic flow will gradually transfer all the load to the stiffer lower layers (*b-c*).

In some cases the compressible top layers may be consolidated by the driving of piles. The piles increase the bearing capacity of the upper layers so that it is necessary to drive them only far enough to reach the stiffer lower layers, as shown in Fig. 1(b). The procedure in such soil is to start driving piles at widely spaced intervals and then to drive intermediate ones until the bearing capacity, as indicated by the rate of penetration, is increased to the desired degree.

Class 3. *Soil which consists of a deep deposit of fairly uniform consistency.* The load-carrying capacity of such soils is becoming better understood due to recent investigations. The ability to carry load depends, in a large measure, upon the ratio of the width of the foundation to the length of the piles. Where the length of the piles is equal to or greater than the width of the foundation, the piles are beneficial, since the load is spread over an area larger than that of the foundation. If the piles are shorter than the width of the foundation, the load is not spread over a greater area, hence no appreciable benefit is obtained. In computing the load-carrying capacity of a foundation on soils of this type it is not permissible to rate them as equal to the load capacity of a single pile multiplied by the number of piles. Under such conditions if greater load capacity is needed, there are the alternatives of increasing the foundation area or using longer piles.

Further discussion of soil behavior under pile loads is given in Section IV—Load Capacity of Pile Foundations.

Foundation Exploration

Before the foundation can be designed, it is necessary that the soil characteristics at the site be known. Sometimes, due to previous experience with existing structures, reliable information is already available. If not, a special exploration survey should be made.

The extent of and equipment for such a survey will depend on the size of the project and the time available. Small projects may be explored with simple, homemade devices, while large projects may warrant elaborate equipment and the services of a geologist. Shallow foundations may be investigated by means of a sounding rod, earth augers or wash borings.

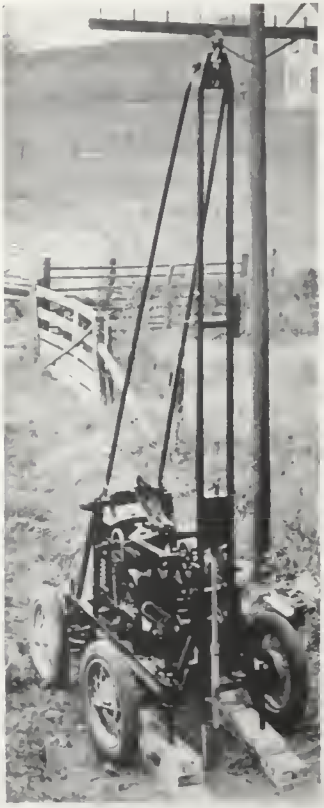


Fig. 2—Calibrated power-operated sounding rod developed by Ohio State University and the Ohio State Highway Department.

The number of tests should be sufficient to obviate misleading conclusions from such purely local effects as the striking of boulders or finding of an old filled-in water course. A test hole should penetrate sufficiently into hard strata to determine their thickness and to prove that the underlying support is adequate.

Soundings

The sounding rod is usually a ⅝ to ⅞-in. round steel rod, pointed at one end and threaded at the other to receive additional lengths. It is driven into the ground by a maul or sledge and turned after each blow. Soundings 30 to 35 ft. deep can be made in this way, but the nature of the material penetrated cannot be accurately determined. The relative hardness of the strata may be judged by the action of the rod during driving. In sand, gravel and clay there will be some penetration under each blow. If rock is reached, there will be no penetration and the rod will quiver and make an entirely different sound under the blow. To avoid mistaking a large boulder for bed rock, further soundings should be made nearby. Too great reliance should not be placed upon the sounding rod tests because it cannot penetrate hard strata of any material thickness.* The sounding rod is useful in conjunction with other methods of foundation exploration and especially useful for investigating between test piles.

A calibrated sounding rod has been developed by a cooperative investigation of Ohio State University and the Ohio State Highway Department.** A small power-operated drop hammer, Fig. 2, is used to drive a sectional sounding rod to depths as great as 80 ft. To calibrate this device, it was set up at various bridge sites where piling had been driven and a comparison made between the driving energy required and penetration of the test rod, and the energy input and penetration of the piling. Comparisons

*See Reference No. 2 (Numbers refer to bibliography).
**See Reference No. 3

were also made with static load tests on the piles at the various locations. The machine has been used to estimate the lengths of piles required at a number of bridge sites in Ohio. In nearly all cases the total length of piling finally driven was within five per cent of that estimated.

Borings

To investigate shallow foundations, earth augers may also be used (Fig. 3). Dry materials are excavated by the auger alone, but materials which will not stand without support require a pipe casing. Some soils adhere to the bit while others require a bailer or sand pump.

The operation of churn drilling with a hollow drill having a cutting bit at its lower end is called wash boring. Material is brought to the surface by a stream of water flowing under pressure through the hollow drill. Samples are taken by catching this material and allowing it to settle in a bucket. Wash borings may be made in sand, gravel, clay and even hard-pan. They show the type of material penetrated but not its compactness. It is, therefore, possible to misinterpret the test. To eliminate this uncertainty it is advisable to take undisturbed samples of soil by means of a special device attached to the drill. A log of each boring taken may be kept and the results of driving and loading tests summarized, as shown in the chart, Fig. 4. Given the load at failure by load test for several piles as indicated at the tops of the piles, and the loads by impact or pile driving formula shown at the side of each pile near the ground line, a quite accurate interpretation can be made of the load capacity of the soil penetrated.

An informative and profusely illustrated discussion of equipment for foundation exploration may be found in "Highway Bridge Surveys" by C. B. McCullough, *Technical Bulletin No. 55*, U. S. Department of Agriculture.

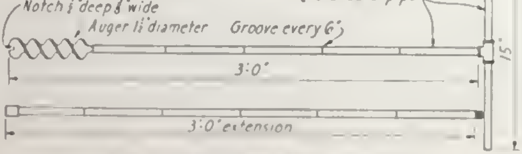


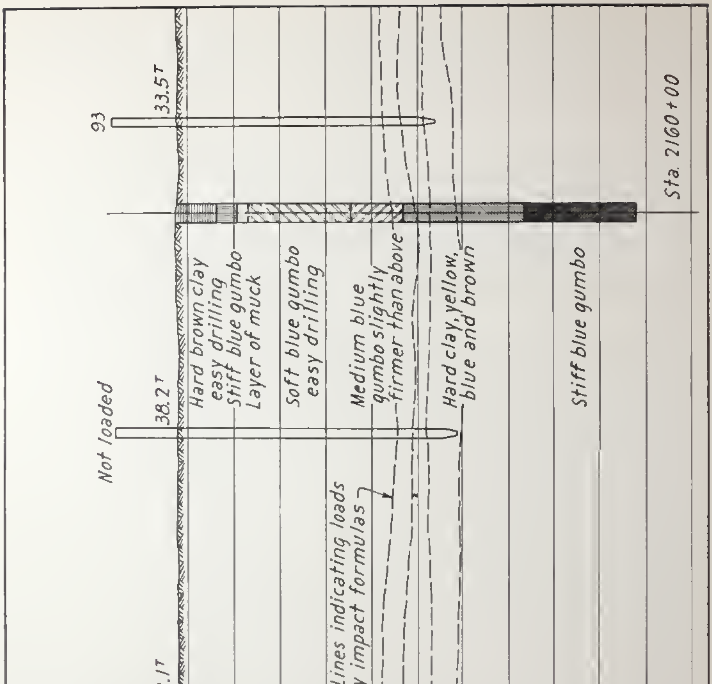
Fig. 3 —Earth auger and extension.

Other Considerations

The location of the permanent ground water elevation should be noted as well as the presence of unusual chemicals in the soil, such as acids, alkali, organic matter and the like.

Test Piles

It is common practice in designing pile foundations to assume an allowable load per pile and to provide a sufficient number of piles to carry the expected load. The ability of each pile to carry the assumed load is accepted if the penetration finally achieved under the blows of the hammer is in accord with a pile driving formula for this loading. Whenever doubt arises



Load at failure by load test
Loads by impact formula
103ᵀ
42.7ᵀ
103
32.1ᵀ
Not loaded
38.2ᵀ
93
33.5ᵀ
El.00
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-110
Hard red & blue top clay
Stiff blue gumbo
Stiff blue gumbo going from soft to medium in density
10ᵀ
20ᵀ
30ᵀ
40ᵀ
Hard clay
Hard blue gumbo
Very stiff blue gumbo
Sta. 2155+00
Lines indicating loads by impact formulas
Hard brown clay easy drilling
Stiff blue gumbo
Layer of muck
Soft blue gumbo easy drilling
Medium blue gumbo slightly firmer than above
Hard clay, yellow, blue and brown
Stiff blue gumbo
Sta. 2160+00

Fig. 1

as to the suitability of such a formula under the local conditions, the load-carrying ability of the piles may be determined by static load test.*

Pertinent information helpful in design and construction can be obtained by driving test piles rather than by relying upon an unsupported assumption. A thorough study was made, for example, to determine the effect of the cost of the piles on the design of the structure of the Bonnet Carre Spillway Highway Bridge.** Preliminary comparative designs were made for several combinations of span lengths and number of piles per bent, and bids were taken. Test piles were driven and loaded and the information obtained was used in selecting the most economical length of pile, the number of piles per bent and the length of span. The most economical design was that having six 20-in. square concrete piles per bent with spans of 38 ft. 9 in. (see Fig. 5). The piles varied in length up to 90 ft. The maximum design load per pile, including live load and dead load, was 43 tons.

Fig. 5—Bonnet Carre Highway Bridge.

Another typical example of the use of test piles is illustrated in the design and construction of the James River Bridge near Norfolk, Virginia.*** Borings and timber test piles furnished sufficient information to decide upon the general type and layout of the structure and for a preliminary estimate of the cost. To secure more reliable information upon which to order pile lengths (piles up to 115 ft. in length were required) concrete test piles were driven at several places in advance of the trestle construction. These piles were identical with those to be used in the trestle and were driven with the same hammer as the trestle piles. A casting schedule was based on

*Cognisance must be taken of the type of soil when interpreting the load capacity of a single pile as an indication of the capacity of a group of piles as explained in Section IV

**See Reference No. 5.

***See Reference No. 6.

the information obtained. As a result, out of all the piles cast, only one remained unused.

For a foundation covering a large area, it is well to drive test piles at frequent intervals; from the lengths obtained a contour map may be drawn from which to determine lengths of intermediate piles. For long trestles the lengths of test piles may be used to plot a profile to be used in a similar manner.

Pile driving data should be carefully recorded. These data are particularly useful in studying the load tests and the information secured from the foundation exploration survey.

Test piles and driving equipment should preferably be identical with that intended for the final construction. However, this is not always practicable. By establishing a proper basis for comparing the load capacity of piles of different size, smaller piles and equipment may be used for test purposes and reasonably accurate predictions can be made as to the length and behavior of larger and heavier piles.* The relative effectiveness of the hammer blow in driving light and heavy piles as given in Table 3, page 53 may serve as a guide to the choice of hammer to be used with test piles. The Missouri Pacific Railroad successfully determines the required length of 24-in. diameter concrete piles by the penetration obtained on timber piles driven with a hammer comparable with the weight of the test pile.* When comparing piles of different materials, shapes and sizes for load capacity, it is important that the test piles be driven in the same location and that they penetrate the same strata.

Load Tests

Load tests on piles are of two general types:

(1) DYNAMIC LOADING, as represented by pile driving and

(2) STATIC LOADING in which loads are applied slowly and remain in place for various intervals after the pile has developed full contact with the soil.

Dynamic Loading

The behavior of test piles is observed under blows of a falling weight. The dimensions and material of the pile, the weight and fall of the ram, and the cushion and hood used on the pile head are noted. The penetration under the blows of the hammer is recorded throughout the driving and from the penetration for the final blows the load-carrying ability of the pile is computed by some pile driving formula.

A discussion of pile driving formulas and their limitations is given in following sections. While there is some question as to the accuracy of predicting the load-carrying capacity of a pile by a formula, the driving of test piles does give quite satisfactory information for ordering piles of proper length.

Static Loading

Static loads are applied in successive increments and the corresponding settlements measured. A graph based on the data obtained produces a load-settlement curve such as shown in Fig. 6. The graph may resemble a

*See Reference No. 7.

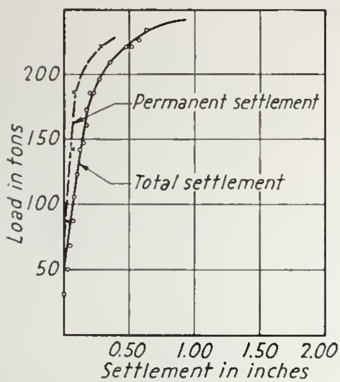


Fig. 6

stress-strain curve having a yield point. If so, the load at the yield point may be considered as the load capacity of the pile. If there is no break in the curve, an arbitrary limiting value of the settlement, such as 0.25 in., may be selected. The allowable design load can then be assigned some proportion of this capacity load, such as 50 per cent.

The total settlement consists of elastic and inelastic or permanent settlement, both of which should be measured. When hydraulic jacks are used to apply the load, the permanent settlement may be determined for each increment of load. At given loads the total settlement is observed, then the load is released and the amount of permanent settlement recorded. When weights are used, the permanent settlements are measured as the load is removed.

The proportion of "yield point resistance" to be chosen as the design load depends upon the judgment of the engineer or upon the job specifications. For example, the specifications of the American Association of State Highway Officials provide that the design load shall be taken as 50 per cent of the load which causes a permanent settlement of ¼ in. in 48 hours.

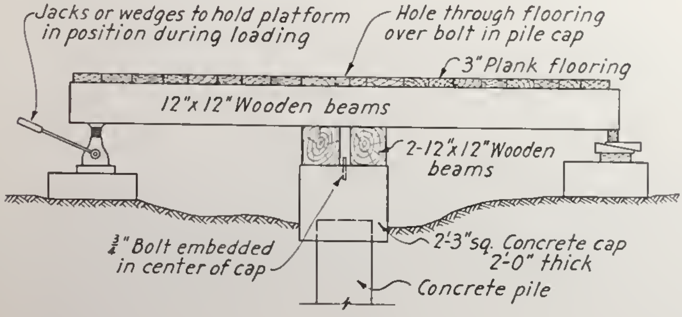


Fig. 7

Methods of Loading

The simple equipment consisting of a platform* required for loading test piles with weights is shown in Fig. 7. The concrete cap cast on top of the pile provides bearing for the 12x12-in. timber beams which support cross beams and plank flooring. A $\frac{3}{4}$-in. bolt on which to take levels is embedded in the cap. Wedges or jacks are placed at the outer ends to

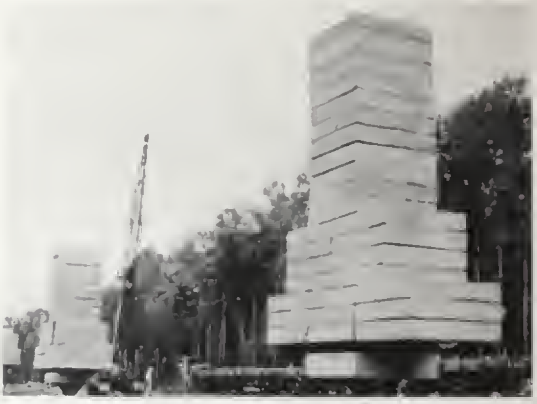


Fig. 8—Testing piles for highway bridge over Bonnet Carre Spillway.

prevent the platform from tipping during loading. After the loads have been placed, the platform is balanced and the blocking lowered slightly so that the entire weight rests on the pile. If sand or other loose loading material is used, the platform must be provided with sides. The head of the $\frac{3}{4}$-in. bolt should be left accessible for taking level readings.

The following method was used in testing piles for a highway trestle near San Francisco, Calif.** Wood piles were driven in a circle around the test pile and a steel tank, capable of holding 100 tons of water, was placed on the pile cluster. During the test the full load of the tank was transferred by jacks to the concrete pile in the center.

Fifty test piles were driven and loaded for the highway trestle across the Bonnet Carre Spillway*** near New Orleans. A heavy steel stirrup was hung on the head of the test pile and a square 5-ton block, having a square hole in the center, was slipped over the pile and rested on the stirrup as shown in Fig. 8. Additional blocks were used for loading.

The Missouri Pacific Railroad employs an ingenious method for testing piles for concrete railway trestles.**** These trestles replace existing timber trestles, which remain in service until the bridge deck is placed. The trestle bents consist of three 24-in. octagonal concrete piles. The center pile of a

*See Reference No. 8.
**See Reference No. 9.
***See Reference No. 10.
****See Reference No. 7.

Fig. 9—The Missouri Pacific Railroad tests 24-in. concrete piles with the aid of a locomotive crane.

bent, about midway between two existing timber bents, is selected for the test. The timber stringers are loosened on the adjacent timber bents and are wedged up on cross timbers over the test pile. A large locomotive crane is then positioned on the trestle so that its front truck is centered over the pile, as shown in Fig. 9. By picking up a known weight at a given

Deck construction on the Reynolds Channel Bridge built by the Long Island State Park Commission. Reinforced concrete deck slabs supported on bents of precast concrete piles make economical, enduring structures and eliminate fire hazard.

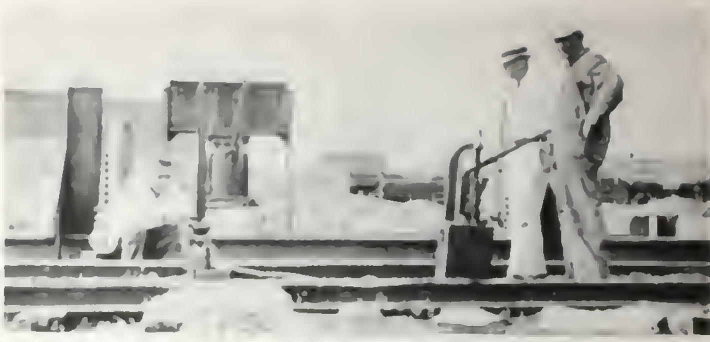


Fig. 10 Testing piles with the hydraulic jack.

boom angle or lifting radius, the load on the pile is readily computed. Settlement of the pile is read with a level from a scale pasted on the side of the pile.

Loading with hydraulic jacks makes it possible to release the load when desired to determine permanent settlement. Anchor piles, a loaded platform or some other device must be provided to take the reaction of the jacks. In Fig. 10 the yoke is fastened to anchor piles.

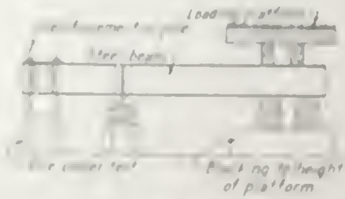


Fig. 11

Pull-out Tests

Pull-out tests are made to determine the frictional resistance of the pile. The difference between the pull-out test and the bearing test is assumed to be the point resistance. A simple type of equipment for making pull-out tests is shown in Fig. 11. In making these tests it is important that the pull be in line with the axis of the pile.

SECTION III — PILE DRIVING FORMULAS

A succession of blows from a falling ram (weight) causes a pile to be driven into the ground to the desired penetration. The penetration per blow during the last stages of the driving is a measure of the load-carrying capacity of a single pile as determined by a pile driving formula. In spite of the limitations of such formulas under certain conditions, they are

widely used because they afford a simple method of determining the approximate capacity of each pile as it is driven.*

The rational basis for pile formulas is the relationship between the energy of the falling ram and the work necessary to drive the pile. If W is the weight of the ram and H the height of its fall, the driving energy available is $W \times H$. Assuming that R is the resistance of the soil and S is the penetration per blow or "set," the work necessary to drive the pile is $R \times S$. If all the energy of the ram is delivered to the pile, a simple relationship exists, namely

$$RS = WH \text{ and } R = \frac{WH}{S} \qquad (1)$$

This is one of the early pile driving formulas and was used by Major Sanders of the U. S. Engineers. This formula does not account for various energy losses and other uncertainties. Major Sanders used the factor of safety of 8 in applying it to timber piles driven into river mud.

Engineering News Formula

A. M. Wellington of the *Engineering News*, 1888, introduced an additional factor C to allow for losses of energy. Then by expressing R and W in tons, S in inches and substituting $12h$ for H (h being the height of fall in feet), and applying a factor of safety of 6, he obtained the well-known Engineering News Formula:

$$R = \frac{2Wh}{S + C} \qquad (1A)$$

For timber piles driven with a drop hammer, a value of 1.0 was assigned to the factor C to give:

$$R = \frac{2Wh}{S + 1.0} \qquad (2)$$

Later, for single-acting steam hammers, the value of 0.1 was assigned to C to give:

$$R = \frac{2Wh}{S + 0.1} \qquad (2A)$$

For double-acting steam hammers the Equation (2A) is applicable by adding the energy developed by the steam to the energy from the fall of the ram. Tables showing manufacturers' energy ratings for double acting steam hammers are given on page 52.

For driving heavy piles with light hammers, the above formulas were found unsatisfactory and further modifications have been suggested to include the ratio $\frac{P}{W}$, where P is the weight of the pile and W the weight of the ram.** The modified formula is:

$$R = \frac{2Wh}{S\left(1.0 + 0.1 \times \frac{P}{W}\right)} \qquad (2B)$$

Energy Losses

Energy is lost in the driving equipment, in inertia of the pile, in heat generated due to impact between the ram and the pile head, and in tem-

*See References No. 35, 36, 37 and 38.
**See Reference No. 11.

porarily deforming the cushion, cap and ground. Several proposed formulas take these losses into account.

When the ram, moving at some velocity, strikes the pile at rest, a certain portion of the energy of the ram is delivered to the pile, propelling it downward against the resistance of the soil. The ram and the pile are semi-elastic and their action after impact conforms to the theory of semi-elastic impact of bodies as given in textbooks on mechanics.*

The elastic properties of bodies are indicated by the *coefficient of restitution, e.* For fully elastic bodies, this coefficient is 1.0 and for inelastic bodies, zero. To illustrate, if a ball drops on a pavement from a height h_1 and bounces to a height h_2, e is given by the relation $h = e^2h_1$, or

$$e = \sqrt{\frac{h_2}{h_1}}. \qquad (3)$$

The percentage of the energy of the hammer transferred to the pile is the efficiency of the blow and is denoted by k. The equations for k in terms of W, P and e are:

For W greater than Pe (no rebound of the hammer),

$$k = \frac{W + Pe^2}{W + P}. \qquad (4)$$

For W less than Pe (hammer will rebound and reduce the energy transferred to the pile),

$$k = \frac{W + Pe^2}{W + P} - \left(\frac{W - Pe}{W + P}\right)^2. \qquad (5)$$

The energy used to temporarily compress an elastic body is given by the area under the load-strain curve. For fully elastic bodies, this curve is assumed to be a straight line, as shown in Fig. 12. The strain or deforma-

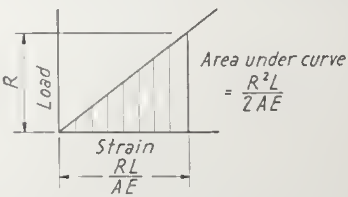


Fig. 12

tion in the pile at the load R is $\frac{RL}{AE}$, where L is the length of the pile, A the cross-sectional area and E the modulus of elasticity of the concrete. The loss of energy in temporarily compressing the pile is therefore:

$$\text{Energy loss} = \frac{R^2L}{2AE}. \qquad (6)$$

Terzaghi Formula

Dr. Charles Terzaghi, in the article "Science of Foundations"** discusses the subject of semi-elastic impact as related to pile driving. He obtains the following formula, which is referred to hereafter for convenience as

*See Reference No. 28.
**See Reference No. 1.

the Terzaghi Formula, by equating the available energy, kWH, to that of the pile resistance, RS, plus the loss of energy in compressing the pile as expressed by Equation (6):

$$WH\left(\frac{W + Pe^2}{W + P}\right) = RS + \frac{R^2L}{2AE} \quad \ldots \ldots \quad (7)$$

Solving,

$$R = \frac{AE}{L}\left[-S \pm \sqrt{S^2 + WH\left(\frac{W + Pe^2}{W + P}\right)\cdot\frac{2L}{AE}}\right] \quad \ldots \ldots \quad (8)$$

Equation (8) is the basic equation for many pile driving formulas. For perfectly inelastic impact $e = 0$ and by substituting in Equation (8) Redtenbacher's Formula is obtained which is used extensively in Europe:

$$R = \frac{AE}{L}\left[-S \pm \sqrt{S^2 + \frac{2W^2H}{E(W + P)}\cdot\frac{L}{A}}\right] \quad \ldots \ldots \quad (8A)$$

Assuming no loss from compression, perfectly inelastic impact and no rebound in the pile, we obtain:

$$R = \frac{W^2H}{(W + P)S}, \quad \ldots \ldots \ldots \ldots \quad (8B)$$

which is the Dutch Formula used extensively in Holland, Belgium and France. Note that when $S = 0$, then R = infinity, which is obviously false and indicates that for small penetrations per blow the result obtained is not significant.

If $e = 1$, Equation (8) becomes:

$$R = \frac{WH}{S + \dfrac{RL}{2AE}} \quad \ldots \ldots \ldots \ldots \quad (8C)$$

If the last term in the denominator of (8C) is assumed to be an empirical constant C, the formula reduces to the Engineering News Formula as given in Equation (1A).

Terzaghi states that e is usually assumed as 0.5, which would result in values of R between those obtained with the Redtenbacher and Engineering News Formulas.

Hiley Formula

The energy equation according to Hiley* is:

$$kWH = RS + \frac{RC}{2}, \quad \ldots \ldots \ldots \ldots \quad (9)$$

where k = the efficiency of the hammer blow as determined by either Equation (4) or (5), and C is the temporary compression in the pile, driving head and ground.

Solving,

$$R = \frac{kWH}{S + \dfrac{C}{2}} \quad \ldots \ldots \ldots \ldots \quad (10)$$

Values for k for various $\frac{P}{W}$ ratios computed from Equation (4) and (5) in which e = 0.25, 0.40 and 0.50, are given in Table I. These values must be multiplied by an equipment loss factor assumed as 0.8 for drop hammers and 0.9 for steam hammers. The value of e in Equation (4) and (5) will

*See Reference No. 12.

vary with the consolidation of the cushion block, which changes during driving. If wood cushion blocks are used with concrete piles, it is satisfactory to assume $e = 0.25$ for fresh packing and $e = 0.50$ for well-compacted blocks.

The factor C is made up of three parts:

C_1 = temporary compression in the driving head in inches;
C_2 = temporary compression in the pile in inches;
C_3 = temporary compression in the ground in inches.

These factors may be determined experimentally or C_1 and C_2 may be approximated by calculating the deformation for an assumed unit stress in the pile and an assumed modulus of elasticity in the pile and wood packing.

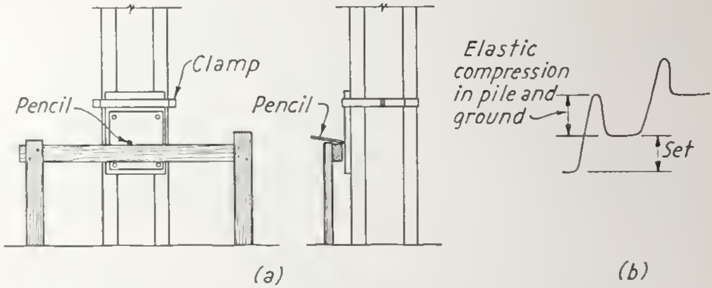


Fig. 13

Experimentally, a simple method of determining C_2 and C_3 is to construct a set gage by clamping a board on the pile to which is fastened a sheet of paper, as shown in Fig. 13(a). In front of this a straight-edge is placed on a frame clear of the pile so that when a pencil is drawn across the straight-edge during driving, a record, as shown in Fig. 13(b) is traced on the paper. From this diagram, the permanent set and the temporary

TABLE I. VALUES OF k
k = efficiency of blow in pile driving

$\frac{P}{W}$ = Ratio of Weight of pile to Weight of Hammer	e = 0 25 Fresh Wood Cushion	e = 0 40 Medium-compacted Cushion	e = 0 50 Well-compacted Cushion
½	0 69	0 72	0 75
1	0 53	0 58	0 63
1½	0 44	0 50	0 55
2	0 37	0 44	0 50
2½	0 33	0 40	0 45
3	0 30	0.36	0 42
4	0 25	0 32	0 36
5	0 21	0 27	0 31
6	0 19	0 24	0 27

To correct for equipment efficiency:
For drop hammers, take 80 per cent of above.
For single-acting steam, take 90 per cent of above.

Sixteen inch (16-in.) octagonal piles, 30 ft. long were used in the foundations for the Texas Building of the Texas Centennial Exposition, Dallas. Piles were cast at the site. A hammer with a 5,000 lb. ram and 36-in. drop was used for driving.

elastic compression below the point on the pile where the record was taken can be measured.

The elastic compression in a driving head having 4-in. packing, assuming a fresh packing with a modulus of elasticity for wood of 50,000 p.s.i. across the grain, is:

$$C_1 = 0.16 \frac{R}{A}, \quad \ldots \quad (11)$$

where R = the ultimate resistance of the pile in tons, A = the area of pile in sq. in.

For well-compacted packing the modulus of elasticity will be higher. Assuming a modulus of 100,000 p.s.i.,

$$C_1 = 0.08 \frac{R}{A} \quad \ldots \quad (12)$$

On the basis of an assumed modulus of elasticity of concrete of E = 2,000,000 p.s.i., the temporary compression in the pile is:

$$C_2 = 0.012 \frac{R}{A} \text{ per lin. ft. of pile.} \quad \ldots \quad (13)$$

The compression in the ground, C_3, is more difficult to estimate, since it varies with the properties of the soil. If possible, it should be measured as previously described. Hiley gives a formula for C_3 in long tons (2,240 lb.) which in terms of short tons (2,000 lb.) is:

$$C_3 = 0.20 \frac{R}{A}. \quad \ldots \quad (14)$$

Actual measurements to determine values of C have been made on piles by Hiley and Ackerman in England and by Goodrich in this country.* Hiley gives the following:

Typical Values of C in Inches for Hard Driving

	Length of Pile in Feet					
	10	20	30	40	50	60
Wood Pile	0.37	.49	.61	.73	.85	.97
Concrete with Short Cushion	.27	.36	.45	.54	.63	.72
Concrete with Driving Cap	.57	.65	.74	.83	.92	1.01

*See Reference No. 13.

Comparison of Pile Formulas

A comparison of the load-carrying capacity of identical piles in the same soil as determined by the Engineering News, Terzaghi and Hiley formulas shows considerable variation, making it necessary to apply factors to them if approximately uniform results are to be obtained. This is illustrated by the following examples, in which:

A = cross-sectional area of 20-in. square pile . . . 400 sq. in.
L = length of the pile . . . . 40 ft.
Hammer (single-acting steam)
equipment, efficiency . . . . 0.90 (For Hiley Formula)
W = weight of ram . . . . 7,500 lb. 3.75 tons
P = weight of pile . . . . 16,500 lb. 8.25 tons
h = fall of ram . . . . 4.0 ft.
H = $12h$, the fall of ram in inches . . . . 48 in.
S = set or penetration per blow . . . . 0.15 in.
R = resistance of pile according to various formulas
E = modulus of elasticity of concrete . . . . 2,000,000 p.s.i.
e = coefficient of restitution
k = blow efficiency from Equation (4) or (5)

By the Engineering News Formula (2A)

$$R = \frac{2 \times 3.75 \times 4.0}{0.15 + 0.1} = 120 \text{ tons.}$$

By the Modified Engineering News Formula (2B)

$$R = \frac{2 \times 3.75 \times 4.0}{0.15\left[1.0 + 0.1\left(\frac{16{,}500}{7{,}500}\right)\right]} = 164 \text{ tons.}$$

By the Terzaghi Formula (8)

$$\frac{AE}{L} = 400 \times \frac{2{,}000{,}000}{40 \times 12} = 1{,}660{,}000,$$

$$WH\left(\frac{W + Pe^2}{W + P}\right) = 7{,}500 \times 48\left(\frac{7{,}500 + 16{,}500 \times 0.5^2}{7{,}500 + 16{,}500}\right) = 174{,}000.$$

Then substituting in equation (8):

$$R = 1{,}660{,}000\left[-0.15 \pm \sqrt{0.15^2 + \frac{174{,}000 \times 2}{1{,}660{,}000}}\right]$$
$$= 1{,}660{,}000\,(-0.15 \pm 0.48).$$

Since the resistance cannot be a negative quantity,

$$R = 0.33 \times 1{,}660{,}000 = 550{,}000 \text{ lb.} = 275 \text{ tons.}$$

In discussing this formula, Terzaghi* does not specify the factor of safety to use. To make the result comparable in this example with those obtained with the modified Engineering News Formula, a factor of approximately 3.5 must be applied.

By the Hiley Formula (10)

For a consolidated wood cushion:

$e = 0.50$, $k = 0.50$ for $\frac{P}{W} = 2$, equipment efficiency 0.90, all from Table I

*See Reference No. 1.

Obtain C_1, C_2 and C_3 from Equations 11, 13 and 14, whence

$$C = (0.08 + 0.012 \times 40 + 0.20)\,\frac{R}{A} = 0.76\,\frac{R}{A}.$$

Substituting in Equation (10):

$$R = \frac{0.9 \times 0.50 \times 3.75 \times 48}{0.15 + \dfrac{0.76 \times R}{2 \times 400}} = 220 \text{ tons.}$$

This is the *ultimate resistance* and should be divided by a *factor of safety* to give the allowable load. Since all the energy losses are considered in Equation (10), a factor of safety of 2 is suggested which gives $R = 110$ tons.

For a Fresh Wood Cushion:

$e = 0.25$, $k = 0.37$, equipment efficiency 0.9, all from Table 1.

C_1 is $0.16 \times \frac{R}{A}$ from Equation (12) and substituting for the other values, those used in the example for the consolidated cushion,

$$C = 0.84\,\frac{R}{A}.$$

Substituting in Equation (10):

$$R = \frac{0.9 \times 0.37 \times 3.75 \times 48}{0.15 + \dfrac{0.84 \times R}{2 \times 400}} = 178 \text{ tons.}$$

Applying a factor of safety of 2, as before, the allowable R is 89 tons

Choice of Weight of Hammer

The weight of hammer required to drive a pile to a specified resistance can be determined by either the Terzaghi or Hiley Formulas. In order to determine the required weight of hammer, select a trial size of hammer and solve for the ultimate resistance at an assumed set. If this resistance is greater than that required, the hammer is of sufficient size.

Applicability of Pile Formulas

Influence of the Ratio $\frac{P}{W}$.

A concrete pile of the same volume as a wood pile will weigh from 3.5 to 7 times as much, depending upon whether the wood pile is yellow pine or oak or one of the lighter woods such as cedar. Concrete piles are usually much larger in section and frequently longer than timber piles so their weight may be from 3.5 to 20 times that of timber piles. Therefore, with the introduction of concrete piles much greater weight had to be handled and driven.

Hammers which had been developed for driving timber piles produced much smaller penetrations per blow when used on concrete piles and sometimes practically no penetration could be achieved. The efficiency of these light hammers was reduced as shown in Table 1. If too light a hammer is used, the capacity of a pile may be overrated by a pile formula of the Engineering News type which does not account for energy losses. When there is any uncertainty as to the proper weight of ram, it is advisable to use a heavier rather than a lighter hammer.

About 50,000 ft. of 24-in. octagonal concrete piles were used on the grade separation project of the Louisville and Nashville RR., Louisville, Ky.

Influence of Types of Soil

When piles are driven, they are forced into the ground by a succession of rapidly applied dynamic loads which keep the ground in an unnatural state. After piles are in their final positions the loads carried are usually static and the ground is in equilibrium. Railway trestles are an exception, as the train load is a high percentage of the total load and is applied to the pile bent as a succession of rolling loads.

A pile formula gives a measure of the capacity of a pile under the rapidly applied dynamic loads but does not necessarily indicate its capacity under static load depending upon the type of soil. Terzaghi* divides soils into two types:

Type A is composed of materials possessing high internal friction without cohesion, such as sand, gravel and permeable artificial fills. The resistance of this type to dynamic loads may, with reasonable accuracy, be assumed as identical to the resistance under static load. *Pile driving formulas may be used with confidence for this type of material.*

Type B is composed of materials possessing cohesion, such as fine grained silts, soft clays and the like. The point resistance during driving is much greater than that later developed under static load, and there is little frictional resistance during driving as compared with that when the pile is at rest. *The application of pile driving formulas to this type of soil is not logical.*

A practical test for determining whether the soil is of Type *A* or Type *B* is to compare the penetration under blows immediately before and after a rest period of at least 24 hours. If they are identical, the soil is Type *A*, and the pile formulas may be used with reasonable assurance.

The water content of the soil is an important factor affecting the reliability of pile formulas. During driving, water from the adjacent soil may lubricate the sides of the pile, particularly in Type *A* soils, which are readily permeable. Where ground water levels fluctuate there may be considerable variation in the soil resistance to pile driving.

*See Reference No. 1.

SECT. IV—LOAD CAPACITY OF PILE FOUNDATIONS

Single Piles

A pile delivers its load to the soil in two ways:

(1) by friction with the soil, and

(2) by point bearing.

The relative percentage of each varies with the soil characteristics and the arrangement of the soil layers. If the point of the pile is driven into a stiff substratum or to rock with softer material above, most of the load is transmitted to the soil at the lower end of the pile. In case the upper strata are very soft as in a Class 1 soil structure (page 6), the pile acts vertically as a column and a comparison of types must be made on the basis of their ability to carry load as columns.

In compressible soils designated as Class 2 (page 7) in which piles are driven simply to compress the soil, thus increasing its bearing value, piles must be compared on the basis of their volumes.

If the pile is driven through essentially compressible material with no stiffer stratum at the point (Class 3 soil, page 7), the pile transfers its load to the soil by increments along its length. These increments depend on the respective frictional resistances and deformations of the strata through which the pile passes. The load is spread in ever widening circles until the point is reached forming what is commonly called a "bulb of pressure." In such soil the surface area of the piles is the basis on which the load-carrying capacity of different types must be compared.

Piles in Groups

The load capacity of a group of piles is not always a multiple of the capacity of a single pile. In Class 1 soils in which the individual pile carries its load as a column, the capacity of a group of piles will equal the capacity of one pile multiplied by the number of piles. In this case, the piles may be spaced as close together as driving equipment permits, since there is no overlapping of bearing areas.

Class 2 soils are elastic and compressible and, as previously described, the purpose of piles driven into such soils is to compact the compressible stratum overlaying a stiffer stratum, thereby raising the bearing value. In a soil of this class, the load capacity of an entire group of piles will be the sum of the capacities of the individual piles, which should be determined from the penetration of the piles by a pile driving formula or by load test after all the piles have been driven. Piles in such soils should not be driven closer together than 2.5 pile diameters.

The bearing value of groups of piles in Class 3 soils, those of uniform consistency, will be limited by the allowable pressure on the subsoil at the bottom of the piles. To obtain this pressure, a certain spread of the load due to the action of the piles is assumed.* The unit pressure determined by this method must be less than the allowable pressure at the given depth. If the subsoil requirements are satisfied, the spacing of piles will be determined by the structural requirements of the footing, provided that the loads on the piles are less than that indicated by a pile formula or a load test. The minimum spacing, center to center of piles, should not be less than 2.5 pile diameters.

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*A draft of the proposed Boston Building Code provides that the computation of pressures shall be made on the assumption that the load from the group of piles is spread uniformly over a horizontal plane, the circumscribing polygon of which extends not more than 0.58 times the length of the piles beyond the edges of the foundation.

Batter piles were used at the outside of each bent to give added stability to the bridge over the Nashua River, Groton-Pepperell, Mass., designed by the Massachusetts Department of Public Works.

Eccentric Loads

When the load on a foundation is eccentric, the piles will receive equal loads if the resultant of the external loads coincides with the center of resistance of the piles as a group. A method for determining the location of piles so that each will be loaded uniformly is given in an article, "Foundation Pile Spacing for Eccentric Loads," by J. W. Pearl.*

Lateral Loads

Batter piles should be used in foundations subject to very large lateral loads. Vertical piles, however, have some resistance to lateral loads if driven sufficiently deep into compact soil. A series of tests** was made at Alton, Ill., on single piles and groups of 4, 12 and 18 piles with their heads fixed in concrete test monoliths. The foundation material was Mississippi River sand and the piles were 30 ft. long and were loaded laterally up to 30 tons per pile. For piling arrangements and the soil under test, it was concluded that the following loads were permissible.

Timber piles for which a deflection of not more than ¼ in. is permitted, 4 tons per pile if the piles are subject to frequent repetitions of load, 4½ tons if subject only to sustained loads.

Timber piles for which a deflection not more than ½ in. is permitted, 6½ tons per pile if piles are subject to frequent repetitions of load, 7 tons if load is sustained.

Concrete piles similar in size, shape and length to the timber piles, 1 to 2 tons per pile more than for timber piles.

For loads within this range the lateral resistance of a group of piles was in proportion to the number of piles in the group. For greater loads per pile and with equal deflections, the total load resisted did not increase in direct proportion to the number of piles. Similar results were found at Red Wing, Minn.***

Marine structures, such as quays and docks, are often subject to large lateral loads and batter piles must be used. It is difficult to drive piles to a

*See Reference No. 14.
**See Reference No. 15.
***See Reference No. 16.

large batter and if a small batter is used a larger number of piles must be provided to develop resistance against lateral forces. An analysis of pile structures of this type is presented in "Resistance of a Group of Piles," by H. M. Westergaard.*

Piles in trestles are subject to lateral loads from wind and the force of moving water. They are also subject to longitudinal loads from traction and braking forces. Trestle bents and bridge piers on piles which extend well above the ground line or are in particularly soft top soil should be designed to resist these forces specified in railway and highway bridge specifications. Consideration should be given to whether the piles resist such forces by simple cantilever action or whether the pile heads are so encased as to be fixed or partially fixed, causing a reversal of bending moments, thus increasing the resistance of the pile to lateral forces. In the latter case, it is particularly important that the pile be designed to resist the stresses developed at the underside of the cap or footing.

SECTION V — PRECAST PILES

Design of Precast Piles

Shape

Piles are usually made of uniform section throughout except near the point. Tapering piles the entire length increases the difficulties of manufacture.

The two most common cross sections for bearing piles are square and octagonal. Round piles are feasible when made by the centrifugal spinning process. Any shape having a radial symmetry is satisfactory.

Square piles have certain advantages:

- They are easier to form and more adaptable to building in tiers.
- The concrete may be placed and spaded into the forms with greater ease.
- The longitudinal reinforcement is more effectively located for resisting flexure.
- There is more surface area per volume of concrete.

Octagonal (or hexagonal) piles have the following advantages:

- They are especially adaptable to metal forms.
- Their strength in flexure is the same in all directions.
- The lateral ties may be in the form of a continuous spiral.
- They present an attractive appearance and any small rotation during driving is not noticeable even though pile is in an exposed position. Special chamfering of the edges is not needed
- They are easily handled.
- Hollow or cored sections may be made readily if conditions warrant.

Size

A marked advantage of concrete piles is that they may be made any size to suit load-carrying requirements, driving conditions and equipment. A large number of small piles or a smaller number of large piles may be chosen, whichever is more economical. Concrete piles from 6 in square to 24 in. square have been precast and driven.

*See Reference No. 17

Length

The length of piles also may be chosen to suit conditions and equipment. In restricted locations or where the handling equipment is light, piles may be made in short sections and extended as penetration is achieved. Such splices may be designed either for maintaining alignment or for developing the full flexural strength of the pile where service loads require.

When desired, piles may be made of great length and handled by special devices. Piles up to 24 in. square and 114 ft. long* have been successfully handled and driven in this country, and precast hollow piles 4 ft. in diameter and 200 ft. long have been used in Europe.**

Reinforcement

The reinforcement in piles is designed to resist the stresses due to service loads and handling and driving. Reinforcement consists of longitudinal bars with hoops or spirals.

Longitudinal bars may be discontinued where not required, but not more than two bars should be stopped off at any place. The sections at which bars are discontinued should be spaced about 30 bar diameters apart. This is necessary because a sudden change in the amount of reinforcement may induce the formation of hair cracks during driving where there is an abrupt change in strength. Splices should be staggered also, for the same reason.

Protective Cover

For bearing piles in which handling stresses govern the amount of reinforcement and where the piles are to be completely buried, the amount of concrete cover is not of major importance. However, when piles are to be exposed above ground, as in trestles and docks, adequate cover for the reinforcing bars is essential. For ordinary outdoor exposures, a cover outside of all bars or ties of 1½ in. of concrete should be provided. For unusually severe exposures, as in sea water or where there are many cycles of freezing and thawing while the concrete is wet, a clear cover of 3 in. is recommended at least for the portion of the pile where such exposure is encountered.

Head

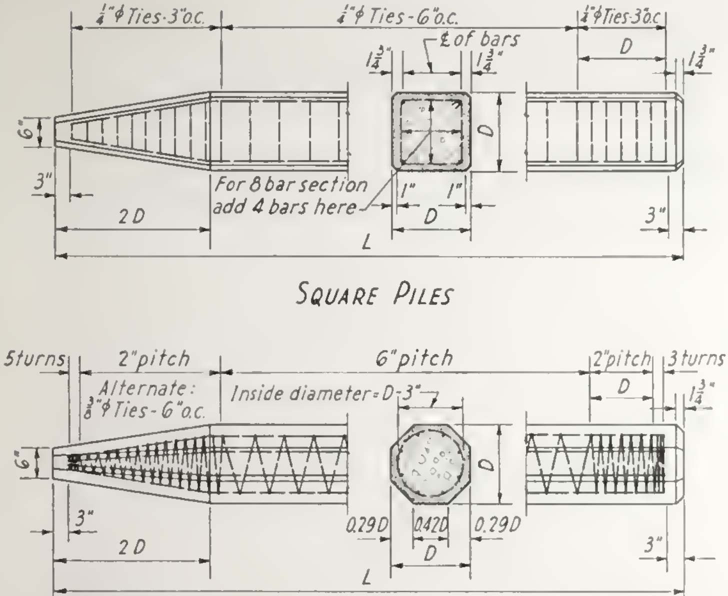
It is important that the head should be designed so it will not be damaged by the hammer blows. To prevent damage, the top edges should be chamfered liberally, the head should be a true plane at right angles to the axis of the pile, and added lateral reinforcement should be provided for a minimum distance equal to the diameter of the pile. Unless the reinforcing bars are to protrude from the head, they should all be kept back about 3 in. from the head for piles 16 in. or more in diameter. (See Fig.14.)

Points

The point of the pile may be adapted to the soil and the method of driving. For plastic soils a blunt point is suited, ranging from no point at all to a diameter at the tip of ¼ the pile diameter and a length equal to 1½ times the pile diameter.

For soils without cohesion, such as sand or gravel, and where hard strata are to be penetrated, a long tapered point is desirable. The tip may have a diameter of ¼ and a length of 3 times the pile diameter.

*See Reference No. 6
**See Reference No. 18



SPIRAL WIRE

D	16"	20"	24"
SIZE	#5	#4	#3

OCTAGONAL PILES

Fig. 14

When concrete piles are properly manufactured, such points may be driven through timber and rip rap; and comparatively large boulders and other obstructions may be displaced. However, it is sometimes desirable to provide a metal shoe for the tip.

If the pile is to be jetted with an internal jetting pipe, the end of the pipe is usually contracted to form a nozzle at the point.

Extra lateral reinforcement should be provided near the point and the ends of the longitudinal bars should be drawn together and should follow the taper of the point as shown in Fig. 14. The bars must not be bunched at one side of the pile.

Stress Determination

Service Stresses

In foundation or bearing piles the service load is almost entirely direct compression. The reinforcement is, therefore, only needed to resist the stresses caused by handling and driving, except in unusual conditions where the pile acts as a column. In trestles and docks, piles subject to lateral loads in addition to direct compression must be designed for bending and direct stress, the same as other structural members.

Bending Moments in Piles Due to Handling

The bending moments in a pile during handling depend upon the method of lifting and the location of the supports. If a pile of length L is supported at points A and B, as in Fig. 15(a), the moment at B is:

$$M_b = \frac{b^2 w L^2}{2}, \quad \ldots \ldots \ldots \ldots \quad (15)$$

where w is the weight per lineal foot of pile.

The maximum moment between A and B is:

$$M_a = \frac{wL^2}{2}\left(1 - \frac{1}{2a}\right)^2 \quad \ldots \ldots \ldots \ldots \quad (16)$$

The moments will be smallest when $M_a = M_b$, in which case $b = 0.293$.

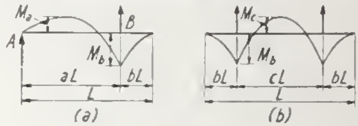


Fig. 15

If the pile is supported at two points equally distant from the ends, as in Fig. 15(b), the reactions will be equal and the moments will be:

$$M_b = \frac{b^2 w L^2}{2} \text{ and } M_c = \frac{wL^2}{2}\left(\frac{c^2}{4} - b^2\right) \quad \ldots \ldots \ldots \quad (17)$$

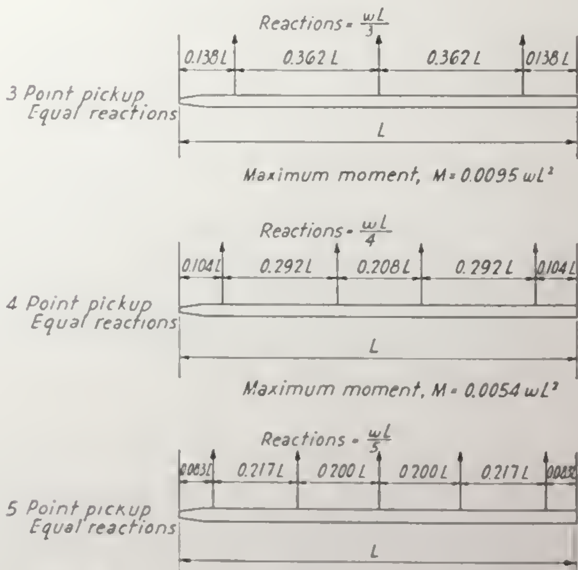


Fig. 16

If $M_b = M_c$, then $b = 0.207$, or the point at which the pile should be lifted in order to secure the minimum moment for this method of support.

For more than a two-point pickup, lifting devices are often arranged so that there will be equal reactions at all points of support, as described in the section "Handling and Driving Piles." The location of pickup points so that bending moments are a minimum for the cases of 3, 4 and 5 supports and equal reactions is shown in Fig. 16. The design bending moments for each type of support are also given. The reinforcement required to resist these moments should be continued throughout the length of the piles, since the moments are nearly equal throughout. If the reactions are not equal, the bending moments for several arrangements of lifting points should be calculated until an arrangement is found which gives nearly the same bending moments at points of support and between the supports.

In determining moments due to handling, piles are regarded as of uniform cross section throughout, the taper at the point being disregarded.

Design of the Pile Section

If only one size and length of pile is to be used, the bending moments are computed for the desired arrangement of lifting points and the pile designed accordingly. If several lengths of piles are involved, the bending moments may be computed for a range of lengths and several arrangements of lifting points, and these values plotted with the resisting moments for various amounts of reinforcement. From such a chart, as illustrated by Fig. 19, the size and number of bars for a pile of any length may be obtained.

In determining the resisting moment of a pile, it is convenient to use the transformed section in which the steel area is considered equivalent to a concrete area n times as large, n being the ratio of the moduli of elasticity of the steel and the concrete. The resisting moment by the well-known flexure formula is $M = \frac{fI}{c}$, in which f is the unit stress at the distance c from the neutral axis, and I the moment of inertia of the transformed section.

The value of I is the sum of the moments of inertia of the elements of the transformed pile section about its center of gravity. The concrete is assumed to have no tensile strength, so only the steel areas are considered on the tension side. The position of the neutral axis is first assumed and the transformed areas and the distances to their centers of gravity are calculated. If it is found that the center of gravity of the section coincides reasonably well with the assumed position of the neutral axis, the moment of inertia may then be calculated about that axis; otherwise a correction must be made.

It is convenient first to calculate I_c, the moment of inertia about the center-line of the pile section, and to determine I by the relation:

$$I = I_c - A_T x^2,$$

where A_T = the total area of the transformed section, and
x = the distance from the neutral axis to the center-line.

The following problem illustrates the procedure in determining the resisting moment of a pile section:

Illustrative Problem

A 20-in. square section is reinforced with four 1-in. round bars, as shown in Fig. 17. Determine the resisting moment of this section when one diagonal is horizontal. Assume $f_c = 1400$ p.s.i., $f_s = 20{,}000$ p.s.i., $n = 10$. Calculate the effective transformed area A, the statical moment Ae, in which e = the distance of the center of gravity of the elemental section from the center-line of the pile section.

As a trial, assume the distance to the neutral axis from the pile center line as 6 in.

	Area, A	Moment Arm, e	Statical Moment, Ae
Concrete	66 2	6 71	576
Steel	7.1	11.66	83
Steel	15 6	0	0
Steel	7 9	—11 66	—92
Total	97 0		567

The distance x from the center-line of pile to the center of gravity of the transformed area is:

$$x = \frac{567}{97.0} = 5.84 \text{ in.}$$

This value of x is sufficiently close to the trial value 6.0 and no correction is needed.* In case there is too large a discrepancy, only the concrete terms used in the calculation need be adjusted.

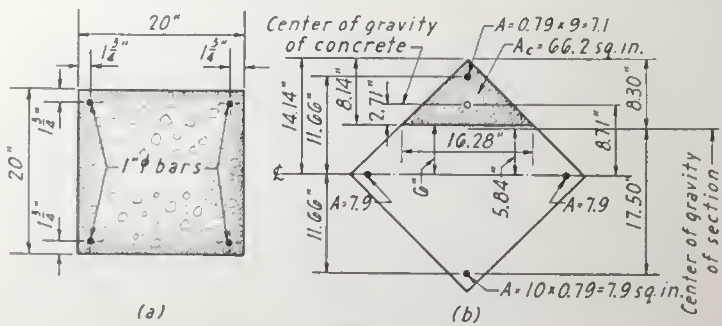


Fig. 17

Moment of inertia of transformed section with respect to its center of gravity is, see Fig. 17(b):

$$\frac{1}{12}(2 \times 8.30)\, 8.30^3 = 790$$
$$7.1\,(11.66 - 5.84)^2 = 241$$
$$2 \times 7.9 \times 5.84^2 = 539$$
$$7.9\,(11.66 + 5.84)^2 = \underline{2420}$$
$$3990 \text{ in.}^4$$

*With the value of 5.84, a re-calculation gives the second value of 5.86 for distance from center-line to center of gravity of transformed section.

The resisting moments of this section are for the concrete:

$$M = 1400 \times \frac{3990}{14.14 - 5.84} = 673{,}000 \text{ in. lb.}$$

for the steel:

$$M = \frac{20{,}000}{10} \times \frac{3990}{11.66 + 5.84} = 456{,}000 \text{ in. lb.}$$

In this case the resisting moment of the steel should be taken as the resisting moment of the pile.

Design Charts for Concrete Piles

Fig. 14 shows typical details for square and octagonal piles, including the arrangement of longitudinal and lateral reinforcing. Fig. 19 gives the size and number of longitudinal bars required for such piles for the three methods of handling shown in Fig. 18, based on f_c = 1400 p.s.i. and f_s = 20,000 p.s.i. without allowance for impact. A protective cover of 1¾ in. to center of steel has been allowed.

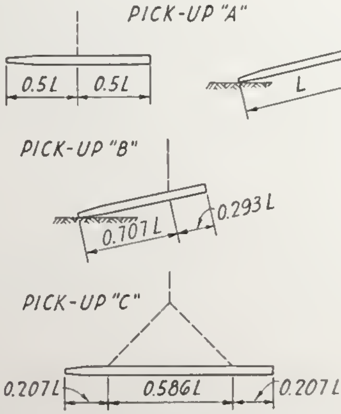


Fig. 18

To use these charts for an allowable steel unit stress of f_s other than 20,000 p.s.i., multiply the ordinate of the resisting moment by $\frac{f_s}{20{,}000}$ where f_s is the new unit stress.

If it is necessary to design the pile for impact, reduce the ordinates of the curves by $\frac{100}{100 + I}$ in which I is the per cent impact.

The following examples illustrate the use of these charts:

1. Determine the reinforcement required for a 20-in. octagonal pile 45 ft. long.

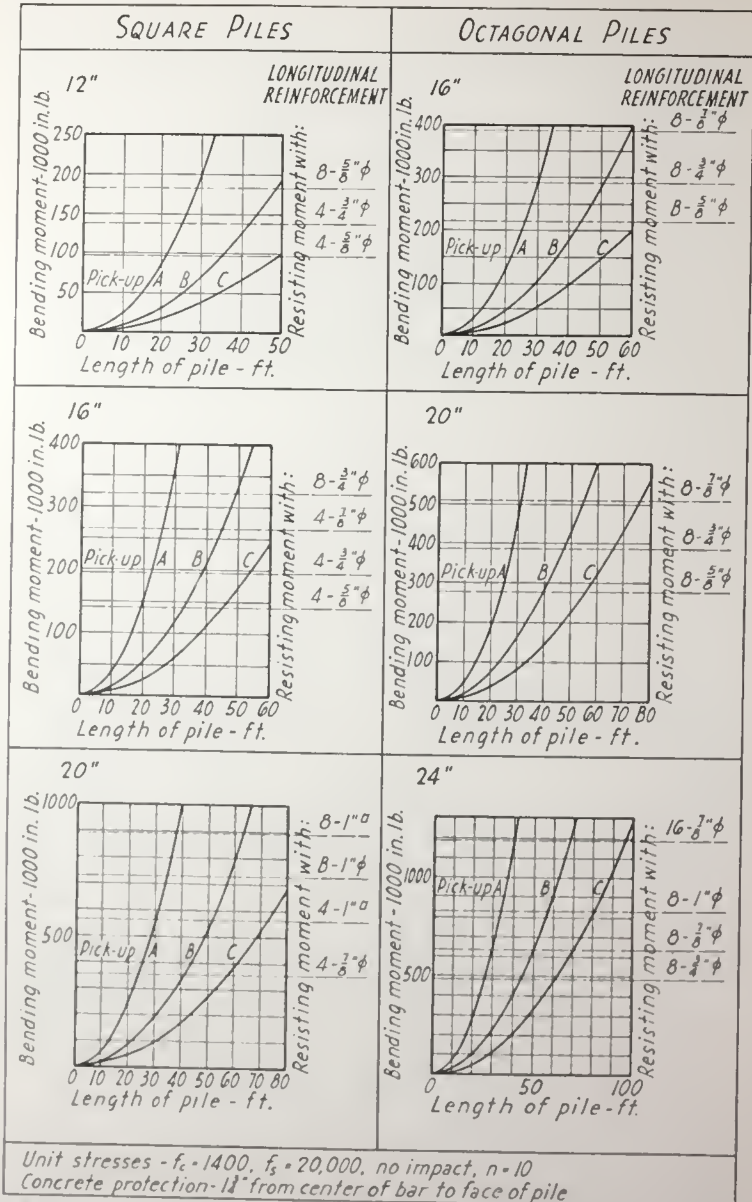

SQUARE PILES
OCTAGONAL PILES
12"
LONGITUDINAL REINFORCEMENT
Bending moment-1000 in.lb.
Resisting moment with:
8-5/8"φ
4-3/4"φ
4-5/8"φ
Pick-up A B C
Length of pile - ft.
16"
8-7/8"φ
8-3/4"φ
8-5/8"φ
8-3/4"φ
4-7/8"φ
4-3/4"φ
4-5/8"φ
20"
8-1"□
8-1"φ
4-1"□
4-7/8"φ
24"
16-7/8"φ
8-1"φ
8-7/8"φ
8-3/4"φ
Unit stresses - f_c = 1400, f_s = 20,000, no impact, n = 10
Concrete protection - 1½" from center of bar to face of pile

Fig. 19

Enter the chart, Fig. 19, at *Length of Pile* = 45 ft. and find the intersection of the vertical line at 45 ft. with lines A, B and C.

For pickup C, at least eight ⅝-in. round bars are required. For pickup B, at least eight ¾-in. round bars are necessary. It is not practicable to lift this pile by pickup A.

2. Determine the maximum pile lengths permissible with pickups A, B and C for a 20-in. square pile with four 1-in. square bars.

The intersection of the dash line for this reinforcement with lines A, B and C gives lengths of 30 ft., 52 ft. and 73 ft., respectively.

If there is some uncertainty about the pickup, the reinforcement may be increased. In all cases it is advisable to mark the pile plainly showing the limits within which the sling should be attached to avoid cracking.

Column Strength of Piles

The formula for tied columns of the 1936 A.C.I. Code* may be used to determine the column strength of piles. This formula is:

$$P = 0.70A_g\,(0.22f'_c + f_s p_g), \quad \text{(18)}$$

where A_g = the gross area of the column;

f'_c = the ultimate compressive strength of the concrete;

f_s = the working stress in reinforcement taken as 16,000 p.s.i. for intermediate grade steel;

p_g = ratio of effective cross-sectional area of vertical reinforcement to the gross area, A_g.

This formula applies for columns having a length h not greater than 10 times the least lateral dimension d. For longer columns the stress is reduced in the following proportions:

$$P' = P\left(1.3 - 0.03\,\frac{h}{d}\right). \quad \text{(19)}$$

A comparison of column strength with bearing power will show that only when very long and unrestrained columns rest on hard strata and develop very high bearing values will it be necessary to design for column strength.

For example, consider a 24-in. octagonal pile having 1 per cent reinforcement and an unrestrained length of 40 ft.

Assume f'_c = 3,500 p.s.i. and

Ag = 477 sq. in.

Then $P = 0.70 \times 477(0.22 \times 3{,}500 + 0.01 \times 16{,}000) = 310{,}000$ lb.

(This gives an average unit stress of 651 p.s.i.)

With an unsupported length of 40 ft. the allowable load will reduce to:

$$P = 310{,}000\left(1.3 - 0.03 \times \frac{40}{2}\right) = 217{,}000 \text{ lb.} = 108 \text{ tons.}$$

Ratios of lateral reinforcement as ordinarily provided are sufficient only for computing the column strength as a tied column. If it is desirable to consider the column as a spiral column, the lateral reinforcement should be increased. In this case the formula* is:

$$P = A_g\,(0.22f'_c + f_s p_g). \quad \text{(20)}$$

The lateral reinforcement required by the A.C.I. Code for spiral reinforcement is:

$$p' = 0.45\,(R - 1)\,\frac{f'_c}{f'_s} \quad \text{(21)}$$

*See Reference No. 19.

Fig. 20—Concrete-paved casting yard.

where p' = ratio of volume of spiral reinforcement to the volume of concrete core (out to out of spirals);

R = the ratio of gross area to core area of column;

f'_s = useful limit stress of spiral reinforcement to be taken as 40,000 p.s.i. for hot rolled rods of intermediate grade and 60,000 p.s.i. for cold drawn wire.

Manufacture of Precast Piles

The fundamentals of design and control of concrete mixtures* recognized as good practice are equally applicable to piles as to all other concrete construction. The quality of the concrete and requirements peculiar to the manufacture of precast piles are given in the section "Specifications for the Manufacture and Driving of Precast Concrete Piles," page 75.

The equipment and facilities provided may either be temporary and "homemade" or the most modern machinery and a layout suitable for long-time operation may be used. Concrete piles may be made either near the site or at a more remote location. Satisfactory piles may be produced under a wide range of conditions if the few simple rules necessary to proper concrete making and construction are followed.

The size and location of the job will determine the location of the casting yard, the layout of its facilities, the type of forms, and the handling and loading arrangements. The topography, transportation facilities, the labor market and other local conditions are to be considered.

The Casting Yard

The layout should be designed to maintain a production schedule. This involves providing storage space for cement and aggregates, mixing capacity, sufficient forms, a casting floor area and space for storing piles before loading that will permit the number of piles to be made and cured properly to supply the job.

The casting floor should be firm to prevent warping or movement of the "green" piles. Soft ground should be compacted and drained and a rigid

*See Reference No. 21.

Fig. 21—Casting yard with embedded timbers and pallet boards.

platform constructed. This platform may either be of timber or concrete as shown in Fig. 20 if continuous, or composed of embedded timbers with "pallet" boards just wide enough for the individual forms as shown in Fig. 21.

A practical and economical platform of concrete may be made of cement bound macadam,* consisting of clean, coarse crushed stone (¾ to 2¼-in.) grouted with a portland cement-sand mixture (1:2¼) in which there is about 8 gal. water per sack cement. The surface is finished by tamping the aggregate and slushing the top with a grout mixture containing somewhat less water than was used for the first grouting.

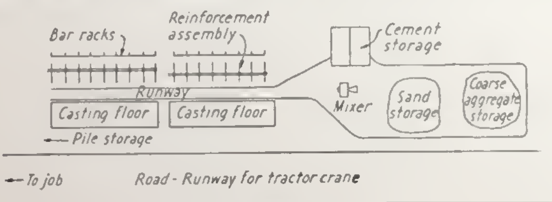


Fig. 22

A suggested layout for a small casting yard is shown in Fig. 22, which provides a mixer with necessary runways, storage facilities for cement, aggregates, reinforcing bars, tables for assembling reinforcing cages, a casting floor, and a storage space for piles before loading. The storage for cement must be dry and it is desirable to plank over the area for aggregate storage to insure clean materials. In case the concrete is obtained from a ready-mix plant, a receiving hopper of a capacity sufficient to hold only

*See Reference No. 20.

the amount of fresh concrete that can be handled readily is provided instead of the space shown for the mixer and storage for cement and aggregates.

The concreting materials in this layout are to be handled by wheelbarrows or buggies. The piles are cast and stored where they can be handled by a mobile crane operated on the adjacent road or track.

Forms

When determining the type of forms and how they are to be fabricated, the size and conditions of the job should be considered and particularly the number of times the forms are to be re-used. The forms for foundation piles that are to be completely buried may be rough and more re-uses of the forms are therefore possible. Such piles, where square in section, may be cast without chamfering the edges, as rough edges are not detrimental.

Forms may be of wood or steel or a combination of the two as, for example, steel body forms and wood points. They should be substantial in construction and should be designed for easy assembling and dismantling to permit re-use. Wood forms when carefully constructed and handled and properly cleaned and well oiled before each use will permit re-using 25 or 30 times. Metal forms may be re-used indefinitely. It is especially important to clean and oil forms thoroughly before putting them in storage.

Forms may be built in sections to give any length desired. The bottom of the form is usually the platform itself or the plank or other pallet on which the piles are cast.

Forms should be tight to prevent leakage and firmly braced so that their position and shape are maintained. When vibration is employed during the placing of the concrete, substantial form construction is particularly important.

A sufficient number of forms should be provided to permit the casting and proper curing of the piles to meet the casting schedule.

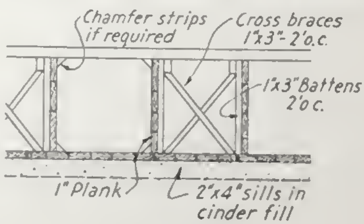


Fig. 23

A practical wood form for square piles is shown in Fig. 23. Chamfer strips may be used as indicated, depending upon whether appearance of the piles in the structure is important.

The form may be assembled in two ways:

(a) Piles are cast as close together as convenient and removed from the platform as soon as they have developed sufficient strength, and the next group of piles is cast in the same location.

(b) The first group of piles is cast on centers twice the width of the pile so that the piles already cast with building paper against them may be used as side forms for the second group. By this method piles may

Fig. 24—Constructing square piles in tiers.

be cast in tiers and a large number cast in a small space. Fig. 24 shows square piles constructed in tiers.

The design for a wood form for octagonal piles is shown in Fig. 25 and metal forms for this type are shown in Fig. 21.

Before placing concrete in the forms, they should be well oiled or, if of wood, may be wetted. Oiling should be done prior to placing the reinforcing.

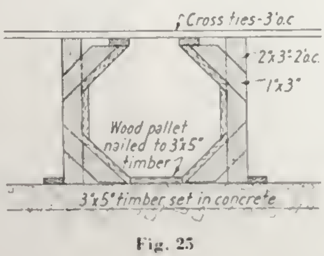


Fig. 25

Concrete

Concrete should be made in accordance with the provision of specifications, page 75, to withstand the abuses of handling, driving and exposure. It should be of a plastic consistency having a 3 to 4-in. slump for hand placing and 1 to 2-in. slump when vibrators are used. A uniform quality of concrete

Fig. 26—Spud type electric vibrator used for compacting concrete for square piles cast in wood forms.

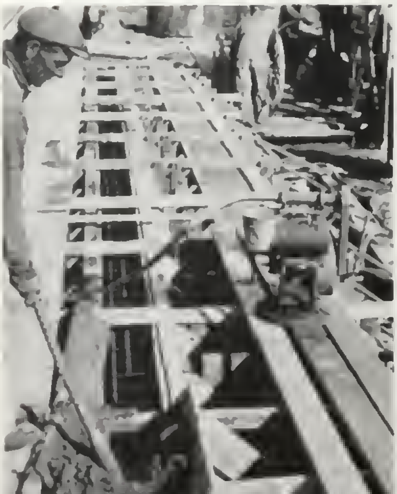


Fig. 27—Stiff concrete being placed with aid of portable gasoline motor-driven vibrator equipped with flexible shaft.

Fig. 28—Electric vibrator shown being used for placing concrete in steel forms for 24-in. octagonal piles.

should be used throughout the length of the piles and special care should be taken to insure well-compacted strong concrete at the head and point.

Vibration* as an aid in the placing of concrete is strongly recommended. The internal method of vibration is most suitable, using high frequency of from 3,600 to 7,000 impulses per min. The use of vibrators is illustrated in Figs. 26 to 28.

One internal vibrator per casting point is sufficient. The vibrator should be inserted into the concrete as it is placed and the form kept as full as practical as the placing progresses from the head toward the point of the pile. The duration of vibration varies with the consistency of the mix but should not be prolonged after the mix has settled and air has ceased to rise. Inserting the vibrator at close intervals for short periods is preferable to prolonged vibration at one location. Placing the vibrator in contact with the reinforcing bars will increase its effectiveness.

To prevent the formation of air voids on the under side of inclined form surfaces, a blade or a loop of ¼-in. round wire attached to the vibrator head should be run along the forms.

The top surface of the pile should be finished with a wood float after the concrete has stood until the surface sheen has practically disappeared. A slight over-filling of the form is desirable.

Curing

Forms should remain in place for at least 24 hours and the piles thereafter should be kept thoroughly wet for at least 7 days if normal portland cement is used, and at least 3 days if high early strength portland cement is used. Ponding is an excellent method of curing if practical. Straw, sand or burlap kept saturated is also satisfactory (Fig. 29). In cold weather, the piles may be covered with tarpaulins and steam, injected through perforated pipe, can be used to supply heat and moisture (Fig. 30). The usual precautions regarding cold weather concreting must be observed.

Whether normal or high early strength portland cement is used, the piles must not be moved until they have acquired sufficient strength to prevent damage. Control cylinders made and cured under the same condi-

*See Reference No. 22.

Fig. 29—Burlap should be thoroughly and continuously saturated throughout the curing period to be effective.

Fig. 30—For cold weather curing, steam injected through perforated pipes under tarpaulins is effective. Curing by this method was used in the casting yard for the L. & N. RR. grade separation project, Louisville, Ky.

tions as the piles are a reliable indication of the strength of the concrete in the piles. Fig. 31 is an illustration of control cylinders and test beams being made on the grade separation project of the Louisville and Nashville Railroad in Louisville, Ky., where approximately 50,000 lin. ft. of 24-in. piles were used. If piles must be moved before attaining full strength for which they were designed, a method of moving or a pickup should be used that will cause stress in proportion to the strength of the concrete.

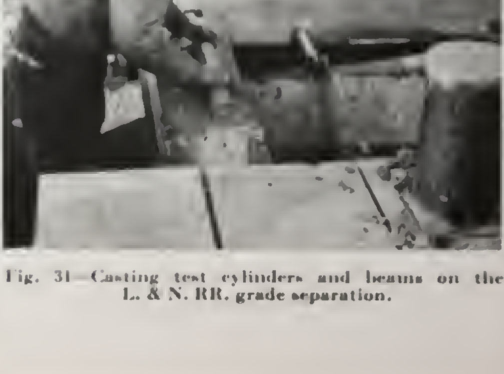


Fig. 31—Casting test cylinders and beams on the L. & N. RR. grade separation.

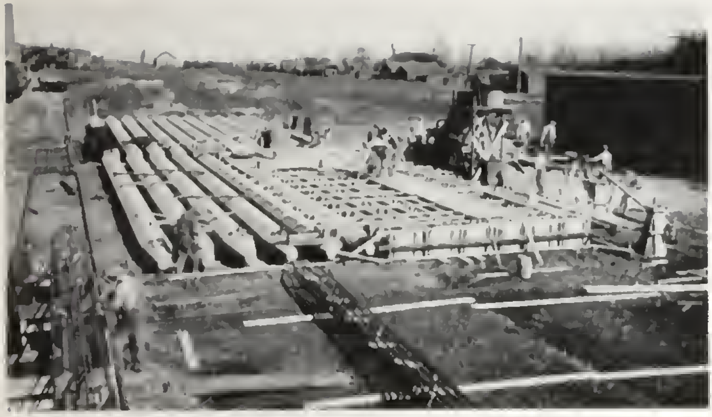


Fig. 32—Reinforcement assembly and pile casting in small yard.

Fig. 33—Reinforcement being moved from an assembly rack.

Reinforcement

It is customary to assemble the reinforcement cages before placing them in the forms, as shown in Figs. 32 and 33. Where the cages are circular, the lateral ties are usually continuous spirals spaced on small steel channels.

The longitudinal bars should end in the same plane and be kept back about 3 in. from the head and the point. Where the bars are drawn together at the point they should not be bunched at one side or the point may be damaged during driving.

The most common oversight in the manufacture of concrete piles is the failure to hold reinforcing bars in their proper positions. Unless firmly supported, the placing of the concrete, particularly when vibration is used, is apt to force the reinforcement cages to one side.

The proper support for reinforcement in the forms before the concrete is placed can not be over-emphasized.

Handling and Driving Precast Piles

In general, piles must be rolled over when removing them from the forms; picked up at the center or at two or more points with an equalizer; loaded; shipped to the site; unloaded at convenient places along the site; picked up and turned from the horizontal to a vertical position; and then driven. Fig. 34 (a) to (f) illustrates progressive steps in the handling and driving of piles for a highway bridge.

The design of the piles and the methods of handling and driving should therefore be studied together.

The reinforcement is often the major cost item and may be considerably reduced by providing proper handling devices to minimize handling stresses.

Fig. 34—Steps in handling and driving concrete piles:

(a) A line from the crane is attached to the head.

(b) The pile is lifted to an erect position.

(c) The point is fitted through a template into a hole in the required location.

A slight excess of reinforcement, however, is sometimes desirable to permit more leeway in handling and driving.

Where many long piles are to be constructed it is sometimes advisable to provide gantry cranes with equalizer lifting devices, spanning a wide yard. For a few small piles, appropriate cranes mounted on caterpillars or rails are usually convenient and economical. The pickup position for which the piles were designed should always be indicated for the guidance of the driving crew. Short piles, 25 ft. and under, are usually designed to be picked up with a single line at any point.

Piles designed for two-point support and lifted by cables, as shown in Fig. 35(a), require a sheave at point A so that the cable will be continuous from point B over the sheave at A to point C. Such a cable is called an equalizer cable because the tension in AB must be the same as in AC. Unless an equalizer is used, the pile must be carefully lifted so that the tension in the cable will be equal and the entire load will not rest on one end. To raise the pile to the vertical, another line CD is attached and when drawn up the sheave at A shifts toward C. A snubbing line is necessary with this cable arrangement as it is with those shown in Fig. 35 (b), (c) and (d) to prevent the pile getting out of control when raising to a vertical position.

A cable arrangement for three-point support is illustrated in Fig. 35(b). The pile is tilted and raised to the vertical the same as for a two-point support. In this diagram, A indicates one end of the cable and B the other, the ends being attached to the blocks. Instead of the two separate blocks as shown at the top, a single block with two sheaves may be used. The vertical components of the reactions on the piles will not be equal because of the inclination of the outer cables. Assuming that they are inclined at

(d) The pile is placed in the leads.
(e) The hammer is fitted on the pile head.
(f) The pile is driven to the required depth.

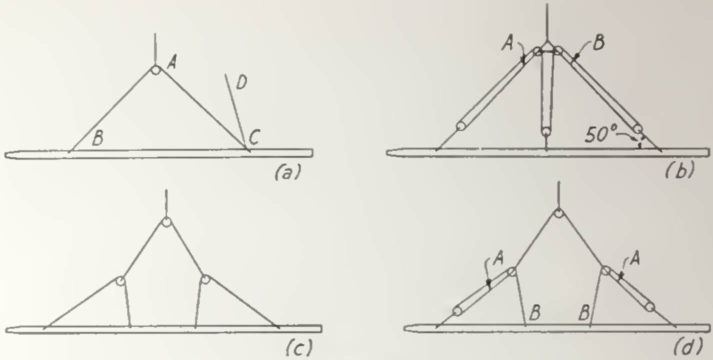


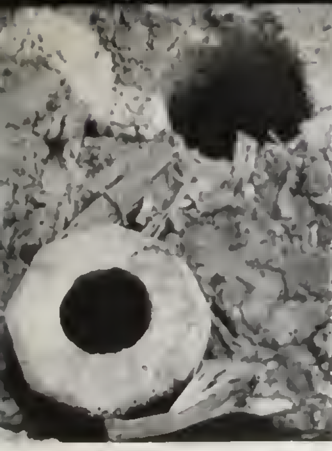
Fig. 35

an angle of 50 degrees to the vertical, the vertical reaction at the outer supports will be sine 50 degrees = 0.76 times that at the center. This should be considered when locating the points of attachment to the pile.

A four-point support may be similar to a three-point support, except that additional sheaves are required at the top and at the points of attachment to the pile. One method is shown in Fig. 35(c) in which the inclination of the end cables is large. To increase the end reactions, the cables may be arranged as shown in Fig. 35(d) in which the ends of the equalizer cables are A and B. Fig. 36 shows a 115-ft., 24-in. square pile being handled with a four-point pickup rig. The cable connection to the head of the pile was used to raise the pile to an erect position.

Photo Courtesy Raymond Concrete Pile Co.

Fig. 36—Rig for handling 115-ft. piles for James River Bridge, Norfolk, Va.



Properly-made concrete piles will withstand the punishment of driving under the most severe conditions without shattering. View above shows large precast piles 40 ft. long driven to penetrations requiring 10 to 12 blows to the last inch. View to the left shows the head of a 24-in. hollow pile 31 ft. long after it has received 2,275 blows from a 9,000-lb. hammer dropping 3 ft. 9 in. at 50 blows per minute.

The possible arrangements for lifting piles by cables and sheaves are almost unlimited. To determine the forces in any suspension system it is necessary to draw a force polygon as shown in Fig. 37. From the given pickup points draw, to scale, a trial arrangement of cables *A*, *B* and *C*. The forces in these cables are equal. To construct the force polygon draw line *a* parallel to *A* and lay off a unit distance *1-2*. Then draw successively *b* parallel to *B* and *c* parallel to *C*, the distances *a*, *b* and *c* being equal. The direction and relative magnitude of *d* is the line *1-4*. Similarly, the magnitude of *e*, half the total weight of the pile, is represented by line *4-5*. The line *1-5* is drawn parallel to the pile. The magnitudes of *a*, *b*, *c*, *d* and *e* are proportional to their lengths in the force polygon. For example assume lengths of *a*, *b*, *c* are 30 in. each, and the lengths of *d* and *e* are 8.4 and 6.8 in., respectively. Then if $e = 5{,}000$ lb., the magnitudes of *a*, *b*, *c* are 2,200 lb. each and the magnitude of *d* is 6,170 lb.

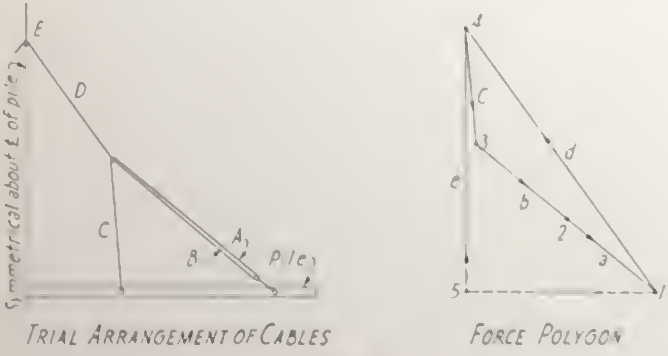


Fig. 37

Photo Courtesy McKiernan-Terry Corp.

A pile driver with fixed leads and double-acting steam hammer. A tractor crane handles the piles.

The arrangement as determined by this method is satisfactory if bending moments in the pile are a minimum for the number of points of support used, and if the height of the sheave at the center is not greater than can be handled by the equipment.

Spreader beams or yokes as shown in Fig. 38(a) may be used instead of equalizer cables for very heavy piles. With them the required height of the crane boom is less and the reactions may be made equal at all lifting points. With a three-point pickup, this is done by adjusting the leverage of the yokes. When a beam or truss is used for this purpose, a continuous equalizer cable is required as shown in Fig. 38(b). Otherwise, the reactions depend upon the initial tensions in the cables and the deflection in the beam or truss. Fig. 39 shows a spreader beam used to carry the sheaves for a three-point pickup for handling sheet piles. The method of transportation, whether by truck, train or boat, should be anticipated in designing piles.

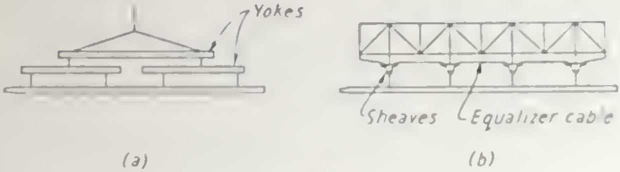


Fig. 38

Loading and Stacking

Where piles are to be loaded or stored in tiers, the blocking between the tiers should be in vertical lines so that the weight of the upper piles cannot produce bending in those of a lower tier, as illustrated in Fig. 40.

Lifting Holds

For ordinary construction, piles may be picked up by cables and hooks looped around the pile at the desired point. However, to protect the edges from damage and to prevent wear on the cable, short lengths of wood or other cushioning material should be used.

Where long piles have been designed and reinforced for definite pickups, hooks or I-bolts are often cast in place with the pile and are cut off after the pile is in position ready for driving. Lifting points are thus precisely located and insure the pile being handled right side up when extra reinforcing bars have been placed on one side. Lifting attachments are sometimes designed to be inserted when lifting the pile, and removed when the pile is in storage or is ready for driving, as illustrated in Fig. 39.

Photo Courtesy Raymond Concrete Pile Co.

Fig. 39—Spreader beam for lifting sheet piles.

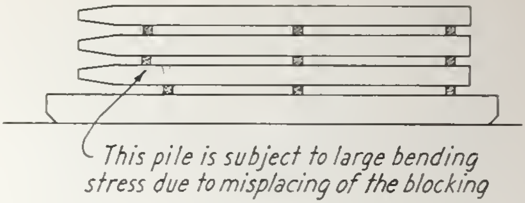


Fig. 40

A typical hold is a 1½-in. I-bolt or T-bolt which screws into a nut and washer cast in the bottom of the pile. To form a hole, a wood pin, substituted for the bolt when the pile is cast, is turned from time to time to keep it loose and is removed when the concrete has set. The hole thus formed is kept plugged until the I-bolt is inserted.

Pile Drivers

The ordinary standard pile driver consists of a high tower of wood or structural steel, to which the "leads" are fastened, and which holds and guides the hammer. A frame supports the auxiliary machinery, consisting of the engine and boiler and various winches and drums to lift the pile and the hammer. This type of driver is mounted on skids or rollers for foundation work, on barges for marine work, and on special cars for railroad work. This type of driver is heavy and cumbersome and is not well adapted for handling piles which must be picked up from some distance away.

Fig. 41—Driving piles with locomotive crane without leads.

Fig. 42 (a) and (b)—Timber framing for guiding piles without leads.

A hammer suspended from a crane mounted on caterpillars as shown in Fig. 34, or from a locomotive crane, Fig. 41, is becoming more popular for constructing trestles and bridges and other foundations. Leads swung from the end of the crane may be used with this type, but with the growing practice of setting and supporting piles in templates the leads are not necessary.

In driving piles for the Havana seawall* a floating timber template fastened to temporary wood anchor piles was used and the hammer was suspended from a crane.

The Missouri Pacific Railroad, in replacing old timber trestles**, supports the piles by two templates, one at the ground and one higher up, as illustrated in Fig. 42(a) and (b). Holes as deep as the points are long are dug through the lower template. A number of piles are then set in position and, by using care in plumbing the pile at the start and keeping the hammer vertical, the piles are driven accurately to position.

Hammers

While concrete piles may be driven with drop hammers, the latter have largely been displaced by steam hammers of the single-acting or double-acting types. In the single-acting type the ram is raised by steam and drops by gravity. In the double-acting type, steam not only raises the ram, but propels it on the downstroke.

The double-acting type delivers more rapid blows which, for the small sizes, may be as many as 300 per min. The weight and energy for different types of steam hammers, as given in the catalogs of several manufacturers, is given in Table 2.

*See Reference No. 26.
**See Reference No. 7.

TABLE 2. PROPERTIES OF HAMMERS

Warrington-Vulcan, Single-Acting Type—Vulcan Iron Works, Chicago—Bulletin 54—[illegible]

No.	Net Weight lb.	Weight of Ram lb.	Stroke ft.	Maximum Energy ft. lb.	Blows Per Min.
6	[illegible]	500	[illegible]	725	[illegible]
3	3,700	1,800	2	3,600	[illegible]
2	6,700	3,000	2 1/2	7,260	70
1	9,700	5,000	3	15,000	60
0	16,250	7,500	[illegible]	24,375	50
*0R	18,[illegible]	9,300	[illegible]	30,225	50

*Special hammer for driving large piles. Not listed in Bulletin 54.

Super-Vulcan, Differential-Acting (*) Pile Hammers
Vulcan Iron Works, Chicago—Bulletin [illegible]—[illegible]

Size	Weight with Base Anvil lb.	Weight of Striking Parts lb.	Normal Stroke in.	Rated Striking Energy ft. lb.	Blows per min.
400	1,887	400	[illegible]	1,025	225
1,100	[illegible]	1,100	9	[illegible]	[illegible]
1,800	[illegible]	1,800	[illegible]	3,600	[illegible]
3,000	7,250	3,000	12	7,260	[illegible]
5,000	[illegible]	5,000	15	[illegible]	[illegible]
8,000	[illegible]	8,000	16	[illegible]	[illegible]
14,000	[illegible]**	14,000	[illegible]	36,000	[illegible]
21,000	[illegible]**	21,000	[illegible]	50,200	[illegible]

*Steam pressure acts on the piston on the up and down stroke similar to a double acting hammer. For full information consult manufacturer's catalog.
**Flat anvil.

McKiernan-Terry Corporation, New York—Double-Acting Pile Hammers—Bulletin [illegible]

No.	Net Weight lb.	Weight of Ram lb.	Length of Stroke in.	Energy ft. lb. Per Blow at Given Strokes Per Min.
9-B-3	7,000	1,600	17	[illegible]
10-B-3	[illegible]	3,000	[illegible]	[illegible]
11-B-3	14,000	5,000	[illegible]	[illegible]
[illegible]*	[illegible]	10,000	[illegible]	[illegible]

*Special hammer for driving large piles. Not listed in Bulletin [illegible].

Union Iron Works, Hoboken, New Jersey—Double-Acting Pile Hammers—Bulletin [illegible]

No.	Average Weight lb.	Weight of Striking Parts lb.	Cylinder Diameter in.	Stroke in.	Energy Per Blow* ft. lb.	Maximum Blows Per Min.
[illegible]	21,000	5,000	14	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1	[illegible]	1,500	[illegible]	[illegible]	[illegible]	[illegible]

*Computed for 80-lb. mean effective pressure.

Industrial Equipment, Cleveland, Ohio—Double-Acting Pile Hammers

Type	Total Weight lb.	Weight of Moving Parts lb.	Maximum Stroke in.	Piston Area Down Stroke sq. in.	Energy Per Blow* ft. lb.	Rated Blows Per Min.
Short Stroke	[illegible]	[illegible]	16	[illegible]	[illegible]	[illegible]
Long Stroke	[illegible]	[illegible]	24	[illegible]	[illegible]	[illegible]

*At 80-lb. pressure.

Table 3 is given as a guide to the choice of hammers to be used with concrete piles. It shows the relative effectiveness of the hammer blow for several sizes of piles and hammers. The table is based on the net energy available for driving a 2,000-lb. pile by a ram weighing 3,000 lb. and delivering 9,000 ft. lb. energy. Equation (4), page 18 was used to determine the efficiency of the blow, e being assumed as zero.

Pile driving formulas and the choice of hammers are discussed in Section III, "Pile Driving Formulas."

TABLE 3. RELATIVE EFFECTIVENESS OF HAMMER BLOWS

$$S = \frac{W}{W + P} \times \frac{E}{5{,}400} \times 100$$

S = relative effectiveness of blow in per cent of that for pile weighing 2,000 lb.;
W = weight of ram;
P = weight of pile;
E = total energy delivered by the hammer.

Weight of Pile—P lb	Weight of Ram and Total Energy		
	W = 3,000 lb. E = 9,000 ft. lb.	W = 5,000 lb E = 17,500 ft. lb.	W = 7,500 lb E = 30,000 ft lb
2,000	100	232 (100)*	440 (100)*
6,000	56	147 (64)	308 (70)
10,000	38	108 (47)	238 (54)
20,000	22	65 (28)	152 (34)

*Reading down, figures in parentheses indicate relative effectiveness for given size of hammer

Fig. 43—Wood cushion for octagonal piles driven with single-acting steam hammer.

Fig. 44—Fitting wood cushion into pile hammer.

Driving Head and Cushion

With steam hammers, a suitable driving head or follower should be provided to fit the top of the pile and to hold the cushion for the pile head, see Fig. 43 and 44. Where pile heads are made with the longitudinal bars protruding, the driving head should be designed accordingly.

The cushion selected depends upon the size of pile, the hammer, hardness of driving and other job conditions. These cushions have been made from various materials including timber, old rope (Fig. 45), belting, various special compositions and even waste and rags. The cushion should give enough protection to prevent damaging the pile head but should not absorb too much of the energy of the blow. The material should withstand the driving of a number of piles to reduce the annoyance of changing and should not be too readily inflammable.

Fig. 45—Rope mat prepared for use on head of 24-in. piles.

Jetting

Concrete piles may be driven by the aid of water or air jets through the same kind of material, usually sandy soils, where jetting for other types is desirable. Pipes discharge water at the point of the pile to erode the soil and bring it up the side of the pile to the ground surface.

The jet pipe may be embedded in the center of the pile and its end constricted at the tip to form a nozzle. Where a center jet is not desired, or where integral pipes have not been provided, external jets may be used the same as for other piles. With a central jet, it may be difficult to hold the pile plumb. Using two or more outside jets, it is possible to correct the position of the pile, since the pile tends to move toward the side on which the jet is discharging. The internal jets are less difficult to handle but are likely to clog in certain types of soil. In gravel, water at high velocity may wash away the fines and leave coarse stones difficult to displace.

In quite porous soils, the water may appear some distance away from the pile being driven, indicating that a passage for the water has not been maintained around the pile. In such a case, the jet pipe should be moved slowly up and down or another pipe provided to discharge above the first.

Piles may be driven in clean sand with the jet alone, but in sandy loam or soft clay a hammer is also required. Jetting is usually discontinued several feet above the final penetration and the pile driven the remainder of the distance with a hammer alone.

If the pile is started by jetting, the long leads or crane boom required for very long piles may not be necessary.

Pumps of adequate capacity should be provided, since volume is more important than pressure. The size of pipe ranges from 1½ to 2½ in. and the size of the nozzle from no contraction to ½ in. The maximum water pressure to be provided varies from 100 to 225 lb. per sq. in., and the volume discharged from 50 to 200 gal. per min.

Alignment

Some tolerance in aligning and plumbing foundation piles which are to be buried should be permitted. However, for trestles and docks, accuracy of line and plumbness should be insisted upon. Wherever accurate alignment and plumbness are required, care in starting the pile correctly will be well repaid. The result of care in aligning piles is well illustrated by a bent of 24-in. piles shown in Fig. 46.

Fig. 46—By starting piles through openings in a template made of structural steel shapes, accurate alignment was secured on the L. & N. RR. grade separations.

Fig. 47(a) to (c)—Steps in cutting off concrete piles:
(a) A sledge and chisel used to remove outer cover of concrete, exposing reinforcing.
(b) An acetylene torch cuts off the bars.
(c) A line from the crane snaps off the head.

The method employed by the Missouri Pacific Railroad*, already described on page 51, located 24-in. piles with an accuracy comparable to that of columns cast in place.

Substantial well-braced frames, such as shown in Fig. 42(a) and (b), are necessary, particularly if the piles must be driven through boulders or other obstructions.

In driving piles close to old bents, walls or piers, they should be started leaning slightly away to prevent the lateral pressure crowding the points over. Forcing pile heads back into line by jacks or similar means should not be permitted unless the ground around the pile is first excavated. Under extreme cases it may be desirable to cut off the piles out of alignment at the ground line and splice on a top portion. Piles may be driven to a batter as great as 3 in 12. Piles on a large batter are difficult to drive and the batter tends to increase.

Cut-offs and Splices

Much misconception exists as to the difficulties of cutting off concrete piles that are too long or extending those that are too short. It is desirable, of course, to order piles of proper length to avoid the waste of material and labor. However, when properly done, the length of concrete piles can be corrected without difficulty after being driven. With pneumatic chisels and

*See Reference No. 7.

drills and the acetylene torch, altering pile lengths becomes simply a routine construction job. To cut off a pile, a V-shaped channel is first cut around the pile at the level of the desired cut-off; the bars are exposed and cut with an acetylene torch at any desired point above cut-off; and then the head is snapped off by wedging or pulling with a line from the crane. The procedure in cutting off a pile is shown in Fig. 47(a), (b) and (c). Cut-offs of large piles, 18 to 24 in. in diameter, can be made in 15 min. if hurried, or 25 to 30 min. normally.

For splicing or connecting to caps or footings, the bars may be stripped for the necessary length (Fig. 48) or holes drilled the proper depth and dowels grouted in. If the pile is to be spliced, a split sectional form may be clamped to the pile, the reinforcing bars set, the top of the pile wet and slushed with grout or mortar, and the concrete placed. The concrete in the splice should be of the same quality as in the driven pile. Internal vibrators will be found effective in making splices.

Fig. 48—Exposing the reinforcing preparatory to splicing a large pile.

Photos Courtesy Raymond Concrete Pile Co.

Fig. 49 (a) to (f)—Procedure in the driving of step tapered shell type cast-in-place piles.
(a) Pile driver, mandrel and unassembled shells.
(b) Shells being placed and screwed together.
(c) Shell completely assembled ready for driving.

SECTION VI — CAST-IN-PLACE PILES

Cast-in-place concrete piles may be made by filling with concrete holes which have been formed in the ground in various ways. They have special advantages for some conditions. Reinforcement is usually not required, unless the soil is such that heaving during construction operations may tend to tear the piles in two, or if they may be called upon to resist service flexual stresses. Extensions and cut-offs are eliminated.

As with other pile foundations, it is important that a knowledge of the soil structure be obtained, and the ability of cast-in-place piles to carry load is based on the same considerations as for precast piles.

Types

Much of this class of work has been done by specialists, featuring various types, methods and equipment. There are three major types:

(1) *Shell Type*—A light metal casing with a steel interior core is driven into the ground to the desired penetration or resistance

(d) **Point of pile.**
(e) **Pile completely driven.**
(f) **Mandrel removed and pile ready for filling.**

in the same manner as precast piles. The mandrel, being collapsible, is then removed and the hole filled with concrete. The shells are tapered and strong enough to prevent serious distortion from lateral pressure. One widely-used type employs a spirally reinforced corrugated shell which comes in sections. The point is 6 in. in diameter and the pile has a taper of 1 in. in 8 ft. A common length for the lower section is 38 ft. with succeeding sections 8 ft. long.

A type recently introduced has a fluted tapered shell with a strong closed point and may be driven without a mandrel.

Before placing the concrete, the holes should be inspected by lowering a light or dropping a flare into them to detect any undesirable deformation of the shell or the presence of water. It is important that water should be pumped from the shell or hole so the concrete is not dropped through water. Placing of concrete should not be started until all shells in a group have been driven and, in general, until all driving within a radius of 15 feet has been completed.

A series of pictures Fig. 49 (a) to (f) illustrates the procedure of constructing shell type cast-in-place piles.

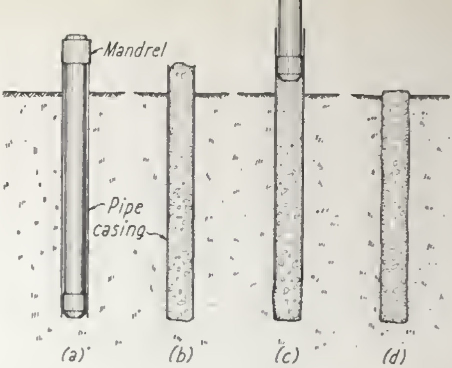


Fig. 50

(2) *Shell-less Type*—A straight pipe is driven with a mandrel which is collapsed and withdrawn when the pile has reached desired resistance. Concrete is placed in the hole and compressed with the mandrel as the shell is pulled up. This forces the plastic concrete into intimate contact with the earth and increases the frictional resistance. If desired, the foot of the pile may be considerably enlarged by pressure to increase the point resistance. The construction of piles of this type is shown diagrammatically in Fig. 50 (a) to (d). There are several modifications of the shell-less cast-in-place pile that have been developed for use in soils of different characteristics necessitating some variations in the methods of construction.

Cast-in-place concrete piles of the shell type were used for this foundation of a heavy retaining wall for track elevation, Rock Island Railroad, Chicago.

Photo Courtesy Raymond Concrete Pile Co.

(3) *Pipe Piles*—are particularly adapted for passing through soft plastic soils to a hard stratum below. They are constructed by driving a heavy metal pipe and removing the earth from its interior with air or water jets, after bed rock or the desired penetration is reached, or, if desired, as the sinking progresses. When bed rock is reached, drilling equipment may be operated through the pipe to remove some rock or to set steel dowels for anchorage.

In some cases a lighter casing is placed inside the heavy pipe and the latter withdrawn, the casing being then filled with concrete. In some wet soils, holes are made by means of a rotary excavator using a bucket with a cutting edge, which is withdrawn occasionally to remove the dirt. As excavation proceeds, sections of metal casing are dropped in place and extended as needed.

Hollow concrete shells of large diameters are often sunk to considerable depths by pressure as the soil is removed from inside them with clam shells. They are later filled with concrete in the same manner as metal pipe piles. Fig. 51 shows designs for hollow concrete piles and illustrates a number of conditions to which they are adapted.

Sectional piles of the pipe type (steel or concrete shells) are especially adapted where headroom is limited, such as for underpinning buildings or working under the decks of bridges and docks. They are also well adapted to locations where the jar of pile driving is objectionable, since they can be driven by jetting or with jacks.

Allowable loads for the types where shells are driven with a mandrel may be based on pile driving formulas the same as precast piles, as discussed in Section III.

Where the walls are formed by excavating or washing out the material, load tests are important. Especially high values may be used for pipe piles driven to rock. For these the New York building code permits a compressive stress of 500 p.s.i. on the concrete and 7500 p.s.i. on the steel in the shell, deducting 1/16 in. from the shell thickness for corrosion.

Concrete for Cast-in-Place Piles

The concrete for cast-in-place piles may be of somewhat leaner mix than for the precast type, since, as a rule, they are wholly buried in the ground and receive vertical load only. However, because of the uncertainties common to all foundation work, and since the extra cost for better quality is slight, it is recommended that not more than 7 gal. water per sack cement be used in the concrete. The mix should be controlled by methods discussed in "*Design and Control of Concrete Mixtures.*"* In placing the concrete, it is important that the holes be dry and the mix be of fairly stiff consistency to avoid water gain. Curing is not usually a factor, since the piles are generally buried. However, during cold weather, where the pile heads project above the frost line the pile heads and surrounding ground should be covered by straw or other suitable protection to prevent frost from damaging the concrete itself or heaving the ground.

*Available free in the United States and Canada on request to the Portland Cement Association.

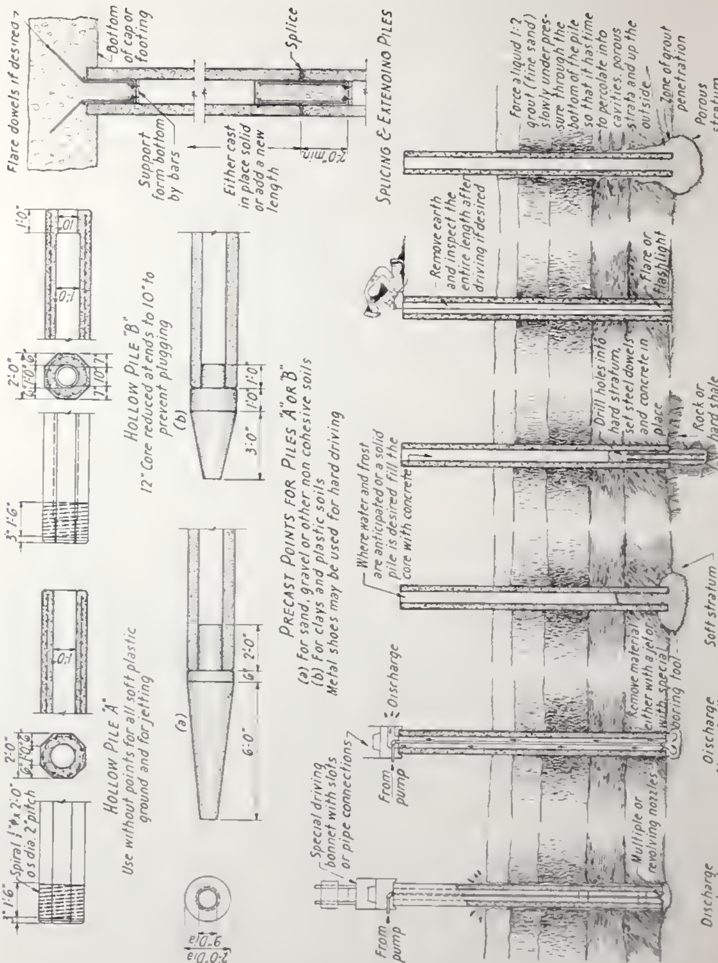


Fig. 51—Hollow Piles and their adaptations.

SECTION VII — SHEET PILES

Shape

Concrete sheet piles are simply precast piles of square or rectangular cross section, driven side by side to form a continuous wall. To keep the piles in line, some form of interlock is needed, such as the tongue and groove joints shown in Fig. 52. The tongue may extend the full length of the pile or for only that portion below the water line, the groove above being grouted to insure watertightness.

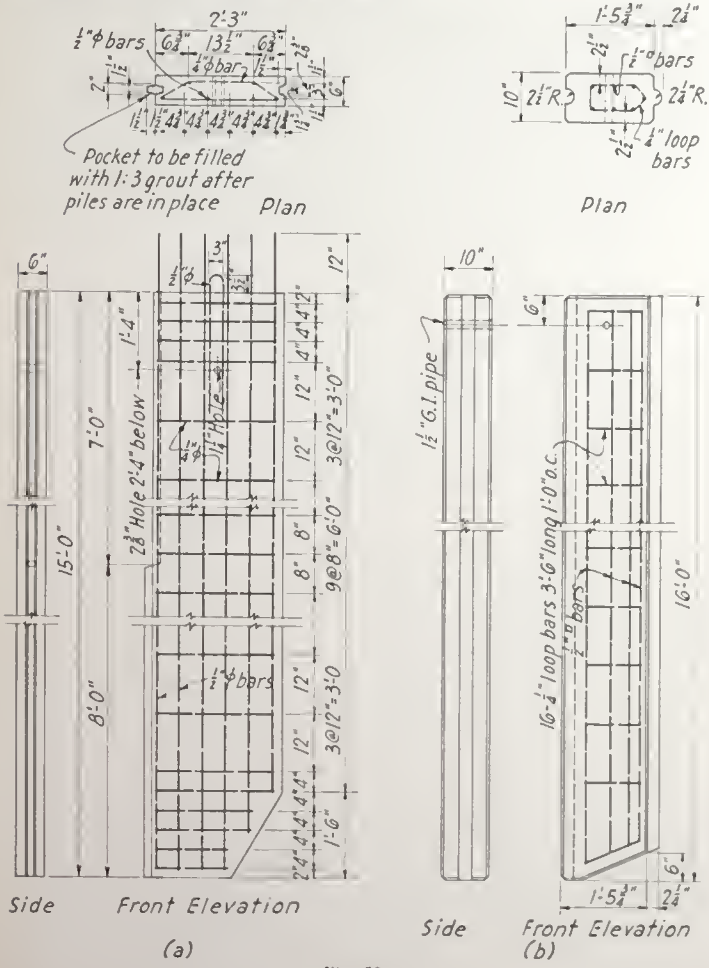


Fig. 52

The foot is usually beveled on one side so that it will be forced against the adjacent pile and maintain contact during driving. The bevel as shown in Fig. 52(a) is better than that shown in Fig. 52(b), because more pressure is developed to hold the piles together. The bevel is shown on the groove side in Fig. 52(a) and on the tongue side in Fig. 52(b). This latter location is better because the pile is driven with the groove side fitting over the tongue of the pile already in place, which prevents clogging. Since the tongue in Fig. 52(a) is only on the lower portion of the pile, the bevel is placed on the groove side. In this case it is not possible to fit the groove over the tongue of the pile in place, since the top of the tongue is below the ground level.

Handling and Driving

Sheet piles must be driven in good alignment and particular care should be given to be sure the first pile of a series is accurately driven. In some soils a timber frame should be provided to keep the piles in line. A typical timber frame having bottom wales braced by stakes is shown in Fig. 53. The frame is moved ahead as piles are driven. Another wale is bolted to the tops of the piles after they have been driven and remains in place until the concrete cap is cast.

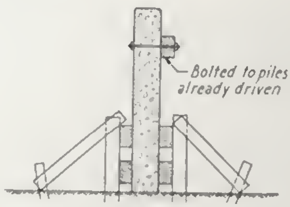


Fig. 53

At changes of alignment, gradual curves may be accommodated by adjusting the framing and using piles of the same section as used for the straight portion. For sharp changes in alignment, it is necessary to provide special angle units.

Since many marine structures are built on sandy soils, jetting is extensively used in driving sheet piles. Jet pipes are cast in the pile, or external pipes may be used. In soils composed of fine sand and clay, it is sometimes possible to wash a hole with the jet below the foot of the pile deep enough to place the pile in its final location. This hole tends to fill up quickly so that speed is required to line the pile correctly and to fit it to the adjacent pile.

Most sheet pile walls must be watertight and joints are usually grouted after the driving is finished. The groove is first flushed out by a water jet from a pipe long enough to reach the bottom of the pile. A cement grout composed of one part cement to two parts sand is then deposited by means of a small sheet metal pipe used as a tremie. The tremie pipe is lowered to the bottom of the hole and then filled with grout. As the tremie is withdrawn and the joint filled with grout, the water is forced out at the top. If the earth fill back of the wall is to be drained, occasional joints may be left without grouting for the lower portion of their length.

To provide for expansion and contraction, joints having flexible fillers may be provided at intervals of 25 to 50 ft., or it may be more convenient to cast a special unit which is solid below ground and split above, the slit being filled with a flexible joint filler. These joints should be continued through the cap.

Sheet Pile Structures

Sheet piles are used principally as cut-off walls to prevent percolation of water and as bulkheads, retaining walls and harbor structures to resist lateral earth or water pressure. A typical cut-off wall for a shore protection structure is that built along the shore of Lake Pontchartrain, La., as shown in Fig. 54. The cut-off wall under the toe prevents waves from underscouring the wall. These piles are not subject to appreciable lateral pressures, hence their size is governed by the conditions of handling and driving.

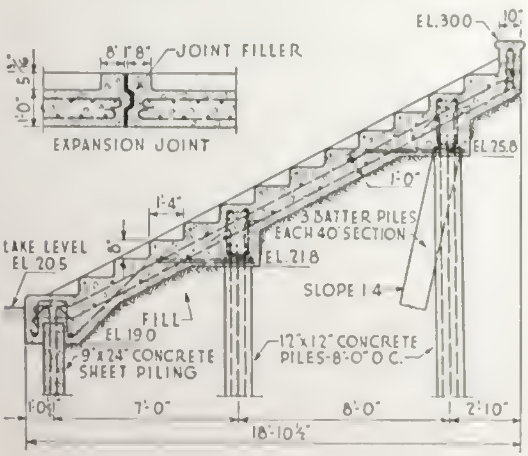


Fig. 54

A sheet pile bulkhead, as shown in Fig. 55, consists of a continuous row of piles driven into firm soil. A cast-in-place cap is connected to an anchor embedded in firm ground. Lateral earth or water pressure is resisted by this anchor at the top and by the earth at the foot of the piles. If this tie is omitted, the resistance of the pile depends entirely on the restraining action of the soil into which it is driven. Sheet piles should be adequately reinforced for bending moments from lateral pressures and should be driven to a sufficient depth so that the soil will not push out at the foot.

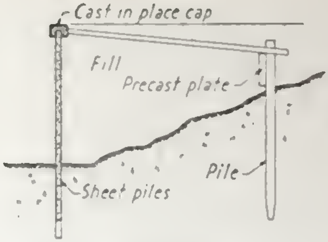


Fig. 55

Details of a small bulkhead type retaining wall, built at Davis Island, Fla., are shown in Fig. 56. This structure consists of precast piles, $6\frac{1}{2}$ in. thick, with a cast-in-place concrete cap. The wall is anchored by reinforced concrete ties 10x12 in. in section to precast plates and single piles spaced 20 ft. on centers.

Concrete sheet piles make efficient docks, jetties, breakwaters and piers. Parallel rows of sheet piles are driven and tied together at the top by encased steel rods, as shown in Fig. 57. The space between the piles is filled with soil or rock and a concrete slab placed on top. To reinforce the structure, diaphragms of sheet piles are placed at right angles to the parallel walls at intervals of about 50 ft.

Where bending moments in sheet piles for high bulkheads are excessive, and to reduce the lateral pressures and consequently the size of the piles, the design may be modified as shown in Fig. 58. A row of piles is driven with their tops at high tide level. A retaining wall is erected above this level with a platform extending back to vertical or batter piles. The weight of the earth fill above the top of the sheet piles is then carried by this platform to the anchor piles so that lateral pressure on the sheet piles is greatly reduced

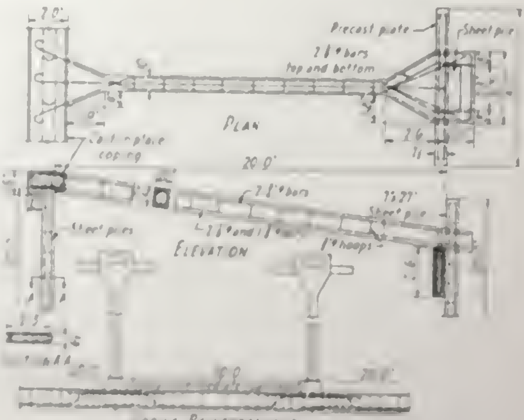


Fig. 56

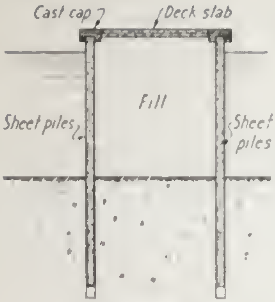


Fig. 57

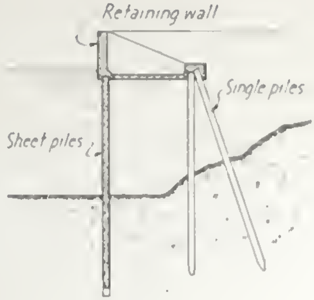


Fig. 58

Another type of bulkhead uses large king piles at regular intervals and smaller and shorter intermediate piles or precast plates between them as shown in Fig. 59. The king piles are more effective in counteracting the overturning of the bulkhead, since they can be driven deeper and are effective over an area greater than the width of the pile.

Design of Bulkheads

Depth of Penetration

The two principal problems in the design of bulkheads are the determination of the pile penetration and analysis of the bending moments due to lateral pressures. The penetration of the piles will be governed in many cases by conditions other than the resistance to be developed to meet lateral pressure, such as the depth required to develop bearing loads, conditions of scour and other foundation considerations. A simple analysis for the penetration required to resist lateral pressure may be obtained by making the following assumptions:

(1) That the maximum active and passive lateral pressures follow Rankine's theory for the lateral pressure of earth.

(2) That the pile is very stiff in comparison with the soil so that the distribution of lateral pressure varies directly as the distance below the surface of the ground.

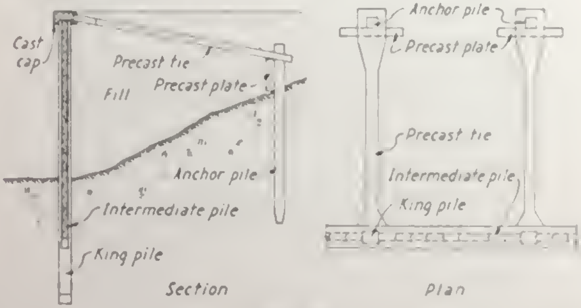


Fig. 59

Rankine's theory gives the following formula for the increment of active pressure:

$$u = w \frac{1 - \sin \phi}{1 + \sin \phi}, \quad \ldots \ldots \ldots \ldots \ldots . \quad (22)$$

in which

w = weight of soil per cubic foot,
ϕ = angle of internal friction.

An increment of pressure is taken as the increase in pressure for points a unit distance apart. If the unit of weight is pounds and the unit of distance is feet, then the unit of u will be pounds per cubic foot and therefore u may be considered as an equivalent fluid weight.

The variation in moisture and compactness of materials causes wide changes in ϕ, as indicated in the following table:

Driving sheet piles for a bulkhead in Lincoln Park, Chicago, Ill., using a water jet. A light drop hammer suspended from hanging leads was used with the water jet for the last few feet of penetration.

TABLE 4*

Material	ϕ
Earth, loam	30° to 45°
Sand, dry	25° to 35°
Sand, moist	30° to 45°
Sand, wet	15° to 30°
Clay	25° to 45°
Gravel	30° to 40°

The weight of soil per cubic foot will vary with the type of material and the saturation of the soil. Soil above the water line will probably not be saturated and that below may or may not be saturated, depending on the tides, the permeability of the soil and the drainage of the wall. The unit weight of submerged soils will be reduced by the buoyancy of the soil particles and may be determined if the percentage of voids is known. The equivalent weight is:

$$w = (\text{weight of dry or moist earth}) - \frac{100 - \%\ \text{voids}}{100} \times 62.5.$$

The pressure on a bulkhead will be the hydraulic pressure plus the lateral pressure of the soil having the reduced unit weight given above. For the angle of internal friction of saturated material, the same value as for the material dry may be used.**

The increment of pressure or equivalent fluid weight for passive pressure by Rankine's theory is:

$$q = w\,\frac{1 + \sin\phi}{1 - \sin\phi} \quad \text{. . . . . (23)}$$

This formula gives the maximum pressure increment that the soil can resist. If the pile is very stiff and does not deflect under load, this pressure will vary in direct proportion to the distance below the lower ground surface as shown in Fig. 60. The magnitude of active and passive pressures is represented by the small arrows in Fig. 60; the resultants, by large arrows, P_P for passive pressure and P_A for active pressure; and the required penetration for the distribution of pressures shown is represented by x. The resultant of all passive pressure will be:

$$P_P = \frac{q\,x^2}{2} \quad \text{. . . . . (24)}$$

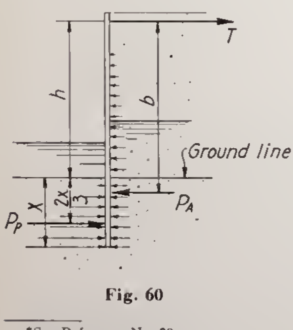


Fig. 60

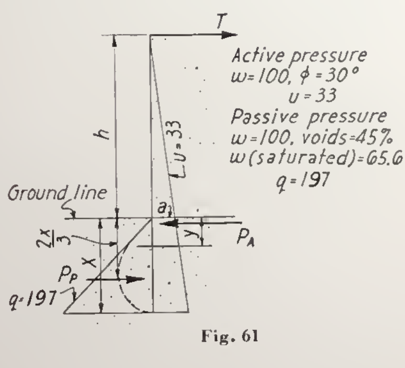


Fig. 61

*See Reference No. 29.
**See Reference No. 30.

Sheet pile bulkhead walls at Davis Island, Tampa, Fla., and elsewhere along the Atlantic and Gulf coast, protect the shore line from erosion.

The value of x may be determined by the relation that the sum of the moments about the tie, T (Fig. 60), must be zero.

$$P_P\left(h + \frac{2}{3}x\right) - P_A b = 0 \qquad (25)$$

Substituting values of P_A and P_P in this equation, the required penetration x may be obtained.

Problem 1

Determine the penetration x in terms of h for the bulkhead shown in Fig. 61. Assume the weight of the soil to be 100 lb. per cu. ft., the angle of internal friction as 30°, and the voids as 45 per cent. The active pressure will be taken as that of dry earth. The sine of 30° is 0.50, and

$$u = \frac{1 - 0.5}{1 + 0.5} \times 100 = 33 \text{ lb. per cu. ft.}$$

For passive pressure the earth is assumed to be submerged so that the pressure is reduced by buoyancy. The equivalent weight of the soil is then:

$$w = 100 - \frac{100 - 45}{100} \times 62.5 = 65.6 \text{ lb. per cu. ft.}$$

The passive pressure is:

$$q = \frac{1 + 0.5}{1 - 0.5} \times 65.6 = 197 \text{ lb. per cu. ft.}$$

The resultant active and passive pressures are:

$$P_P = \frac{197\,x^2}{2} \text{ and } P_A = \frac{33\,(h + x)^2}{2};$$

substituting these values in Equation (25),

$$\frac{197\,x^2}{2}\left(h + \frac{2}{3}x\right) - 33\,\frac{(h+x)^2}{2}\cdot\frac{2}{3}(h + x) = 0.$$

The equation is solved by substituting various values of x until one is found which satisfies, from which $x = 0.55\,h$.

Moments in Sheet Piles

The distribution of passive pressure as assumed in the previous problem is suitable for determining the minimum penetration but is not accurate for determining bending moments. The piles are flexible and, due to the deflection of the embedded portion, the passive pressure is changed from a triangular variation, as shown by the solid line in Fig. 61, to a curve similar to that shown by the dotted line. The latter distribution decreases the anchor tension and the bending moments in the piles. Several methods of analysis* take this variation into account, but many assumptions must be made and the analysis is long and tedious. The following comment was made by M. G. Findley** on this problem:

> "It is clearly a difficult type of analysis because pressures that themselves can be estimated primarily only by methods of indeterminate structural analysis, determine, and in turn are modified by, pressures in which a secondary analysis by means of deformations must be made."

Instead of designing on the basis of several uncertain assumptions, it is convenient for routine design simply to assume the position of the point of zero moment in the sheet piles. Tests and experience with bulkheads show that this point is somewhere below the ground surface and is often taken*** to be at the point of zero pressure. Having assumed the point of zero moment, the sheet pile may be analyzed as a simple beam spanning from this point to the tie.

The distance y from the ground surface to the point of zero pressure is:

$$y = \frac{a}{q - u}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (26)$$

where a = the active pressure at the ground line.

The following problem will illustrate the procedure outlined.

Problem 2

Determine the anchor tension and the bending moments in the sheet piles for the bulkhead in Problem 1. The height of the bulkhead is h = 15 ft., the active pressure increment u = 33, the active pressure at the ground surface $a = 33 \times 15 = 495$, and the passive pressure increment q = 197.

The point of zero moment is:

$$y = \frac{495}{197 - 33} = 3.02 \text{ ft.}$$

The resultant pressure forces and their moment arms are as shown in Fig. 62 (a). Taking moments about the point of zero moment, the tension in the anchor is:

$$T = \frac{3710 \times 8.2 + 747 \times 2.01}{18.02} = 1770 \text{ lb.}$$

The maximum moment in the sheet piles will occur at the point of zero shear. If Z, Fig. 62 (b), is the distance to this point,

$$\frac{Z^2 u}{2} = T, \; Z = \sqrt{\frac{2T}{u}} = \sqrt{\frac{2 \times 1770}{33}} = 10.4 \text{ ft.}$$

*See Reference No. 31.
**_Proceedings_ A.S.C.E., Vol. 62, p. 131
***See Reference No. 32

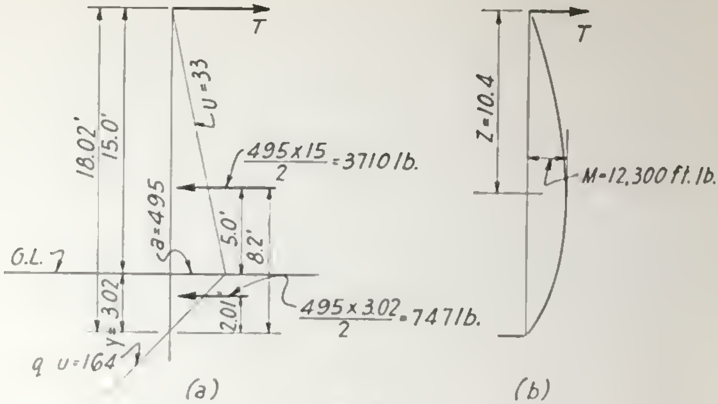


Fig. 62

The maximum bending moment in the pile is:

$$M = TZ - \frac{TZ}{3} = \frac{2TZ}{3} = \frac{2 \times 1770 \times 10.4}{3} = 12{,}300 \text{ ft. lb.}$$

Problem 3

This problem takes into account a variable distribution of active lateral soil pressure and hydraulic pressure.

Determine the minimum penetration and the size of section required for the sheet pile bulkhead shown in Fig. 63. The high and low water elevations, the filling materials and their properties are shown in Fig. 63(a). The fill is assumed to be saturated to a level midway between high and low water. The wall is anchored at the top.

The loading that creates the greatest lateral pressure will occur with the water at low water elevation. For this condition, the pressures on the back of the wall will be increased by hydraulic pressure up to the line of saturation. Unit pressures due to active loads are shown in Fig. 63(b) and the resultants of pressure triangles and rectangles in Fig. 63(c). The net passive resistance in front of the wall is $q - u = 197 - 25 = 172$

The point of zero pressure is $y = \dfrac{581}{197 - 25} = 3.4$ ft

The sum of lateral forces is: $699 + 538 + 274 + 2604 + 450 + 975 = 5540$ lb. and the moment of forces about the tie is: $M = 699 \times 4.33 + 538 \times 7.75 + 274 \times 8.17 + 2604 \times 12.0 + 450 \times 13.0 + 975 \times 16.13 = 62{,}270$ ft. lb.

The distance of the resultant from the tie is the moment divided by the force, $62{,}270/5{,}540 = 11.2$ ft. The additional penetration x can be determined by the relation that the moment of the triangular area of passive pressure about the tie must be 62,270 ft. lb. or:

$$\frac{172\,x^2}{2}\left(18.4 + \frac{2x}{3}\right) = 62{,}270 \text{ ft. lb}$$

Solving by substitution gives $x = 5.7$ ft. The minimum penetration is $3.4 + 5.7 = 9.1$ ft. The depth necessary to prevent scour and to develop vertical bearing must also be considered.

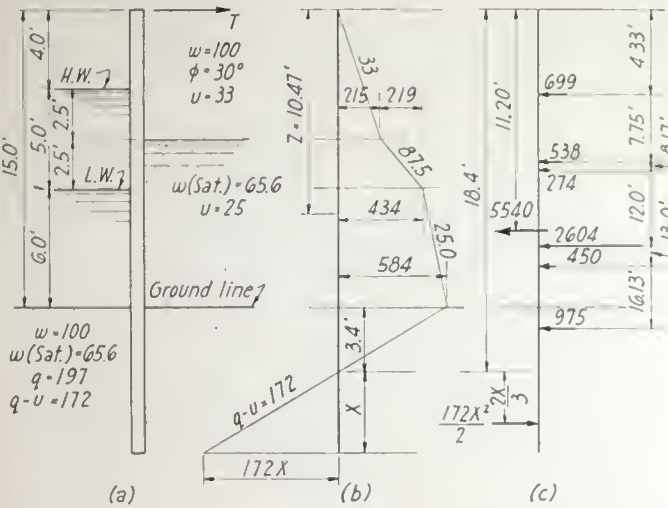


Fig. 63

The tension in the anchor for a simple beam* of 18.4 ft. span will be:

$$T = \frac{5{,}540\ (18.4 - 11.2)}{18.4} = 2{,}170 \text{ lb.}$$

The maximum moment in the sheet piles will occur at the point of zero shear, the point at which the resultant of active pressures equals the anchor tension. By inspecting the load diagram, Fig. 63(c), it may be seen that this point is below the low water elevation, say at a distance of Z from the tie.

Equating the anchor tension and the active pressures:

$$2170 = 699 + 538 + 274 + 434\ (Z - 9) + \frac{25.0\ (Z - 9)^2}{2}$$

from which $Z = 10.44$ ft. The bending moment about this point is:
$M = 2170 \times 10.44 - 699 \times 6.11 - 538 \times 2.69 - 274 \times 2.27 - 434 \times 1.44 \times 1.44 \times \frac{1}{2} - 25.0 \times 1.44 \times 1.44 \times \frac{1}{2} \times 1.44 \times \frac{1}{3} = 15{,}850$ ft. lb.

A section designed for this bending moment is shown in Fig. 64. The reinforcing bars are more than $3\frac{1}{2}$ in. from the compression face and contribute but little as compression reinforcement so that the section may be designed as a beam with only tension reinforcement. Using the value of K the coefficient of resistance $= 249$ for balanced reinforcement which results from allowable unit stresses of $f_c = 1400$, $f_s = 20{,}000$, and $n = 10$, the minimum depth of section d for a 1-ft. width is:

$$d = \sqrt{\frac{M}{bK}} = \sqrt{\frac{15{,}850 \times 12}{12 \times 249}} = 8.0 \text{ in.}$$

Since a protective cover of 3 in. is desirable for sea water exposure, the

*Passive pressure below the point of zero pressure is not included, as it does not affect the beam which is considered simply supported.

depth of section will be taken as 12 in. and the depth d to the tension reinforcement as 8.1 in. The required tension reinforcement is:

$$A_s = \frac{M}{f_s\, jd} = \frac{15{,}850\times 12}{20{,}000\times 0.875\times 8.1}$$

$$= 1.33 \text{ sq. in. per ft. width.}$$

For a section 24 in. wide, two $\frac{7}{8}$-in. round bars and two 1-in. round bars will be required on each side, as shown in Fig. 64.

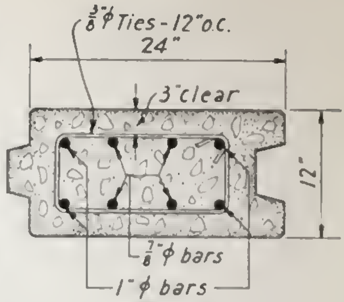


Fig. 64

The reinforcement in this section should also be checked for stresses due to handling in the manner described in Section V, "Precast Piles."

Anchors

Three types of anchors for bulkheads are shown in Fig. 65. The plate shown in Fig. 65(a) depends for its support on the passive pressure of the soil and must be buried a sufficient depth and distance from the wall to

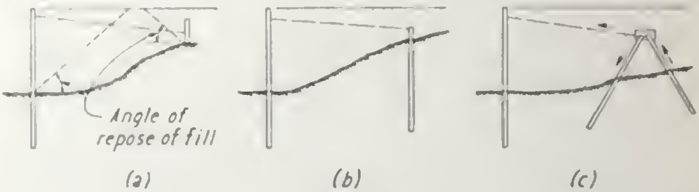


Fig. 65

develop the passive pressure. A line from the bottom of the plate representing the angle of the internal friction of the fill should not cross a similar line drawn from the intersection of the piles with the ground surface, as shown in Fig. 65(a). To prevent settlement, the plate should rest on firm ground and not fill. Otherwise, it should be held in place by bearing piles or single sheet piles. A buried cantilever or counterfort wall may be substituted for the plate and is more stable due to the weight of soil above it. Such a wall may be designed as a retaining wall and is not dependent upon the passive pressure of the soil.

Vertical anchor piles as shown in Fig. 65(b) may be bearing piles or single sheet piles. These should be driven into firm ground for the greater part of their length. Tests to determine lateral resistance should be made on such piles if possible. The results of such tests are shown on page 26.

Batter piles, Fig. 65(c), are more stable than vertical piles. They show less deflection, since resistance is developed by direct stress in the piles as shown by the arrows. The batter pile at the right is subjected to pull and should be tested for this condition if possible.

Pedestal piles with a bulb at one end were used in the Ocean Beach Esplanade*, San Francisco, Calif., to develop resistance to pull-out. They were driven by jetting.

*See Reference No. 33.

SECTION VIII — SPECIFICATIONS FOR MANUFACTURE AND DRIVING OF PRECAST CONCRETE PILES

Materials

1. Portland Cement

Portland cement shall conform to the "*Standard Specifications for Portland Cement*" (*A.S.T.M. Serial Designation: C150-41*) and shall be Type ___.

These specifications cover five types of portland cement as follows and provide that "when no type is specified, the requirements of Type I shall govern"*.

"Type I.—For use in general concrete construction when the special properties specified for Types II, III, IV and V are not required.

"Type II.—For use in general concrete construction exposed to moderate sulfate action, or where moderate heat of hydration is required.

"Type III.—For use when high early strength is required.

"Type IV.—For use when a low heat of hydration is required (Note).

"Type V.—For use when high sulfate resistance is required (Note).

"Note.—Attention is called to the fact that cements conforming to the requirements for Type IV and Type V are not usually carried in stock. In advance of specifying their use, purchasers or their representatives should determine whether these types of cement are, or can be made available."

2. Concrete Aggregates

(a) Concrete aggregates shall conform to the "*Standard Specifications for Concrete Aggregates*" (*A.S.T.M. Serial Designation: C33*).** Where aggregates conforming to these specifications are not obtainable, aggregates that have been shown by test or actual service to produce concrete of the required strength, durability and watertightness may be used where authorized by the engineer.

(b) Maximum size of aggregate shall be not larger than 1½ in. nor more than three-fourths of minimum clear spacing between reinforcing bars.

3. Water

Water used in mixing concrete shall be clean, and free from deleterious amounts of acids, alkalis, or organic materials.

4. Metal Reinforcement

(a) Metal reinforcement shall conform to the requirements of the "*Standard Specifications for Billet-Steel Bars for Concrete Reinforcement*" (*A.S.T.M. Serial Designation: A15*), or for "*Standard Specifications for Rail-Steel Bars for Concrete Reinforcement*" (*A.S.T.M. Serial Designation: A16*).

(b) Cold-drawn wire for concrete reinforcement shall conform to the requirements of the "*Standard Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement*" (*A.S.T.M. Serial Designation: A82*).

5. Storage of Materials

Cement and aggregates shall be stored at the work in such a manner as to prevent deterioration or intrusion of foreign matter. Any material which has deteriorated or which has been damaged shall not be used.

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*These paragraphs including Note are quoted from above specifications.

**Where reference is made to A.S.T.M. Standards and the year of adoption is not shown, the current standard shall apply.

Concrete Quality

6. Concrete Quality

The concrete shall contain not more than 6 gal. water per sack cement, including the surface moisture carried by the aggregates. Piles subjected to sea water or other severe exposure shall contain not more than $5\frac{1}{2}$ gal. water per sack cement. Moisture in the aggregates shall be determined by methods which will give results within 1 lb. per 100 lb. aggregate, and proper deduction shall be made in the amount of water added to each batch.

7. Tests on Concrete

(a) During the progress of the work, compression tests shall be made in accordance with the *"Standard Method of Making and Storing Compression Test Specimens of Concrete in the Field"* (*A.S.T.M. Serial Designation. C31*). Each test shall consist of one laboratory control cylinder and one field control cylinder.

(b) At least one test shall be made for every twenty-five (25) piles cast and not less than one test shall be made for any one day's operation.

(c) The standard age of test shall be 28 days, but 7-day tests may be used, provided that the relation between the 7 and 28-day strengths of the concrete is established by test for the materials and proportions used.

(d) In all cases where the average strength of the laboratory control cylinders shown by these tests falls below an ultimate compressive strength of 3500 p.s.i., the engineer shall have the right to order a change in the mix or in the water content for the remaining piles. In cases where the average strength of the cylinders cured on the job falls below the required strength, the engineer shall have the right to require conditions of temperature and moisture at the job necessary to secure the required strength.

(e) In the event that the engineer changes the water content specified, adjustment, covering amount of cement and aggregates affected, will be made as an extra or a credit under the provisions of the contract.

8. Concrete Proportions and Consistency

(a) The proportions of aggregate to cement for any concrete shall be such as to produce a mixture which will work readily into the corners and angles of the forms and around reinforcement with the method of placing employed on the work, but without permitting the materials to segregate or excess free water to collect on the surface. The combined aggregates shall be of such composition of sizes that when separated on the No. 4 standard sieve, the weight passing the sieve (fine aggregate) shall not be less than 30 per cent nor greater than 50 per cent of the total unless otherwise required by the engineer.

(b) The methods of measuring concrete materials shall be such that the proportions can be accurately controlled and easily checked at any time during the work.* Measurement of materials for Ready-Mixed Concrete shall conform to the *"Standard Specifications for Ready Mixed Concrete"* (*A.S.T.M. Serial Designation C94*).

(c) The slump of the concrete shall be not greater than 4 in. when concrete is placed by hand, or 2 in. when placed by vibration.

*Wherever practicable the engineer should require measurement by weight rather than by volume.

Mixing and Placing Concrete

9. Preparation of Equipment and Place of Deposit

(a) Before placing concrete, all equipment for mixing and transporting the concrete shall be cleaned, all debris and ice shall be removed from the forms, which shall be thoroughly wetted (except in freezing weather) or oiled, and the reinforcement shall be thoroughly cleaned of ice or other coatings.

10. Mixing of Concrete

(a) The concrete shall be mixed until there is a uniform distribution of the materials and shall be discharged completely before the mixer is recharged.

(b) For job-mixed concrete, the mixer shall be rotated at a speed recommended by the manufacturers and mixing shall be continued for at least one minute after all materials are in the mixer.

(c) Ready mixed concrete shall be mixed and delivered in accordance with the requirements set forth in the *"Standard Specifications for Ready Mixed Concrete"* (*A.S.T.M. Serial Designation: C94-38*).

11. Conveying

(a) Concrete shall be conveyed from the mixer to the place of final deposit by methods which will prevent the separation or loss of the materials.

12. Depositing

(a) When concreting is once started, it shall be carried on as a continuous operation until the pile is completed, beginning at the head and working toward the point of the pile. The top surface shall be screeded and brushed to a uniform even texture similar to that produced by the forms. No concrete that has partially hardened or been contaminated by foreign material shall be deposited in the forms, nor shall retempered concrete be used.

(b) All concrete shall be thoroughly compacted by vibrating and/or spading and rodding during the operation of placing and shall be thoroughly worked around reinforcement and into the corners of the forms.

(c) The frequency of vibrators shall be not less than 3600 per minute. The intensity of vibration shall be sufficient to cause the concrete to flow and settle into place, and to make the effect on the concrete visible over a radius of at least 2 ft. Vibrators shall be applied at points not over 2 ft. apart and there shall be an average of not less than 20 seconds of vibration per ft. of pile. In general, vibration shall be of sufficient duration to accomplish thorough compaction and complete embedment of reinforcement. To secure even and dense surfaces free from honeycomb, vibration shall be supplemented by spading or rodding by hand while concrete is plastic under the vibrating action.

13. Protecting and Curing

Side forms may be removed 24 hours after concrete is placed, provided the concrete has hardened sufficiently.

Provision shall be made for maintaining the surfaces of concrete made with normal portland cement moist for at least seven days and for that made with high early strength portland cement at least the first three days after the placement of the concrete, or until the concrete has attained a compressive strength of 2500 p.s.i. as shown by test cylinders under like curing conditions.

14. Cold Weather Requirements

(a) Adequate equipment shall be provided for heating the concrete materials and protecting the concrete during freezing or near-freezing weather. No frozen materials or materials containing ice shall be used.

(b) All concrete materials and all reinforcement and forms with which the concrete is to come in contact shall be free from frost. Whenever the temperature of the surrounding air is below 40 deg. F., all concrete placed in the forms shall have a temperature of between 70 deg. F. and 80 deg. F., and adequate means shall be provided for maintaining a temperature of 70 deg. F. for not less than 3 days after placing except when high early strength portland cement or concrete is used the temperature shall be maintained at not less than 70 deg. F. for 2 days, or for as much more time as is necessary to insure proper curing. The addition of salt or other chemicals to the mix for the prevention of freezing shall not be permitted.

Forms and Reinforcement

15. Design of Forms

Forms may be of wood or metal and shall conform to the shape, lines and dimensions of the pile as called for on the drawings, and shall be substantial and sufficiently tight to prevent leakage of mortar. They shall be properly braced or tied together so as to maintain position and shape.

16. Cleaning and Bending Reinforcement

Metal reinforcement, at the time concrete is placed, shall be free from rust scale or other coatings that will destroy or reduce the bond. Bends for ties shall be made around a pin having a diameter not less than two times the minimum thickness of the bar. All bars shall be bent cold

17. Placing Reinforcement

Metal reinforcement shall be accurately placed in accordance with the plans and shall be adequately secured in position by concrete or metal chairs and spacers. A minimum protective cover of 1½ in. of concrete shall be provided over all reinforcing, except that for piles subjected to sea water or other severe exposure 3 in. of concrete shall be provided.

18. Splices and Offsets in Reinforcement

Splices of reinforcement at points of maximum stress shall generally be avoided. Splices shall provide sufficient lap to transfer the stress between bars by bond and shear.

Handling and Driving

19. Marking

Each pile shall be stamped or marked with the date of its manufacture. Lifting points indicated on the drawings shall be plainly marked

20. Handling

Piles shall be handled carefully to avoid dropping or severe jarring while in a horizontal position. Piles shall be handled only when the concrete has attained a strength of 3500 p.s.i. as determined by field control test cylinders as required in Section 7, unless special provision is made for handling to reduce the stresses in proportion to strength of the concrete.

21. Driving Cap

Piles shall be protected with an approved cushion and cap while being driven.

22. Hammer

A steam hammer shall be used unless other equipment is permitted by the engineer. The weight of the striking parts of the hammer shall not be less than one-third the weight of the pile.

23. Leads

Piles shall be secured against lateral movement during driving by leads or other suitable means.

24. Jetting

Piles may be driven with a hammer and water jets or by water jets alone. Jet pipes may be either separate or cast in the pile. An ample supply of water at adequate pressure shall be provided. Piles shall be driven by a hammer alone for the last 5 ft. of penetration.

25. Penetration

Piles shall be driven to depths or to penetrations per blow as directed by the engineer. Accurate records of the penetration per blow for the last foot shall be kept for the guidance of the engineer in determining allowable loads on the pile. Where driving is interrupted before final penetration is reached, the record for penetration shall not be taken until after at least 12 in. penetration has been obtained on resumption of driving.

26. Replacing

Any pile so injured in driving or handling that its structural integrity as a pile under the conditions of use is impaired, shall be replaced by a new pile, or the injured part replaced by splicing or otherwise repaired as directed by the engineer.

27. Alignment

Unless otherwise called for on the plans, piles shall be driven as nearly as possible in plumb position. Any pile so out of line or plumb as to impair its usefulness shall be pulled and redriven or an additional pile driven as directed by the engineer.

28. Splicing

Heads of piles shall be cut off or the concrete stripped from the reinforcement as directed by the engineer. If proper resistance to driving is not attained at contemplated level of cut-off, the driving shall be continued and an additional length of pile required shall be supplied by splicing in such a way as to develop the full strength of the section of the pile.

29. Test Piles

Test piles shall be of the same size and materials as the permanent piles and shall be driven with the same equipment and in the same manner as specified for such pile. Test piles shall be driven in advance of final driving of permanent piles so that lengths for casting may be determined. During driving, an accurate record of the penetration shall be kept. Load tests shall be made with equipment approved by the engineer. Compensation for driving and loading test piles shall be made at a unit price agreed upon in the contract.

BIBLIOGRAPHY